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(Deep Underground Construction)

Volume I

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FOREWORD

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INTRODUCTION TO THE SYMPOSIUM

J. J. O'Sullivan
The RAND Corporation

Because of the widespread current interest in the problems of protecting military installations located deep underground or under mountains, The RAND Corporation, in connection with its work for the United States Air Force under Project RAND, is sponsoring this Second Protective Construction Symposium which will stress primarily the design and construction of underground facilities to resist the effects of nuclear weapons.

The purpose of this symposium is first of all to advance the knowledge of military personnel and architect engineers in this new and important phase of protective construction, so that military and engineering decisions may be made with full knowledge of the problems involved and the solutions currently available. It will also familiarize you with specialized information, small pieces of which are now scattered among many people. The present state of the art of tunnel design will be defined. Tunnel failure mechanism will be discussed, and guidance will be provided on the phenomenology involved in underground installation collapse caused by surface bursts of large-yield weapons.

The RAND Corporation was established in November 1948 as an independent, nonprofit organization. It had its beginning in 1946 when work began on United States Air Force Project RAND, a long-range research activity. RAND research is primarily concerned with the application of new methods of scientific analysis and a multi-disciplinary approach to the solution of problems of long-range planning. The Corporation is divided into a research staff of about 500 technical people working in five divisions: Economics, Engineering, Mathematics, Physics, and Social Science. Each division has its specialized problems, but major RAND studies cross administrative lines and draw upon all the available professional skills of the organization in their solution. At present the Corporation is devoting a major share of its efforts to research on problems related to national security, particularly those of primary interest to the Air Force. A smaller effort is sponsored by the United States Atomic Energy Commission. In addition, the Corporation conducts a program of research with its own funds.

A very small part of the over-all RAND study effort is involved in considering questions of protective construction and related civil engineering problems. For the most part, these are generated by RAND's study of future weapon systems.

We are in the somewhat unique position, because of our close working arrangements with physicists studying future weapons capabilities, of anticipating future shelter requirements some years before the needs become generally accepted by the engineering profession. For this reason we have in the past promoted work in the field of shelter design to resist high overpressures at a time when 10 psi was considered the top military requirement.

Nearly two years ago The RAND Corporation sponsored jointly with the Ballistic Missile Division of the Air Research and Development Command, United States Air Force, our First Protective Construction Symposium, the purpose of which was to determine the state of the art in the field of protective construction, and to point out the degree of protection necessary for the vital elements of air bases and missile bases if these bases were to survive long enough to launch their alert aircraft and missiles. Since our studies had convinced us that fast reaction time was critical, we included papers on the problems of runway debris clearing and the related problems brought about by operations in a fallout environment.

For the past two years we have been actively investigating the need for a small number of superhard deep underground centers designed to resist close-in multiple enemy attacks by high-yield nuclear weapons. From these studies we have been made aware that much of the current tunnel design practice depends largely on intuition and experience rather than engineering principles, and that there is a great need for determining the basic phenomena involved in the loading of deep underground structures by high-yield nuclear weapons.

Because much interest has been developing during the past six months in the possibility of constructing underground shelters and because it is quite possible that structures of this type may be built in the immediate future, we decided to hold this symposium for military officers and civilian architects and engineers who are interested in these problems. Only nonclassified subjects will be considered.

Careful study of the deep underground shelter problem shows that the interaction and layout of equipment, choice and design of utilities, and construction techniques have a serious influence on the over-all design of the structure, which in turn affects the strength and cost of the installation. Possibly no other engineering structure is so dependent upon the proper balance in design of components for a good over-all solution as the deep shelter. For this reason and to give you an appreciation for the complete problem, we have arranged for experts in many varied fields to present papers on special facets of the problem. These will include the need for shelters, weapons effects to be provided against, the interaction of the utilities and structural solutions. Also, experiences will be presented from other fields such as submarine design, oil-well drilling and present day underground powerhouse design. These fields indicate the availability of a vast fund of knowledge, portions of which may be modified and adapted for our use.

WHY GO DEEP UNDERGROUND?

Herman Kahn
Princeton University

Some of you were present at the first RAND Protective Construction Symposium and may remember a talk I gave entitled, "Why Shelters?" In this talk I discussed possible roles that protective construction might play in our two major strategic weapons systems--the manned bomber and the ballistic missile. At that time the whole question of protective construction was extremely controversial. The controversy was not only about the cost and performance of various measures, but about philosophic aspects of offense vs defense doctrines. Discussions often became heated with many references to Maginot-mindedness and ostriches.

Much of that first talk was concerned with trying to show that although the customary Western military bias toward the offense had made sense for the operating commands in the past, it will not be a reliable guide in the future for planning force posture or composition. I am going to say very little today about the merits of the offense vs defense controversy. That debate seems to have been settled in the last few years. In fact, what little I do say about offense vs defense can be construed as leaning more toward the offense than the defense.

Today everybody realizes that measures designed to insure the survival of our strategic forces after a surprise attack by the enemy must be emphasized. Of course, measures designed to reduce vulnerability are quite likely to either cost money or introduce operational complications, but we have made up our minds to live with these costs and inconveniences. There is still much controversy about both the qualitative and quantitative aspects of the necessary defensive measures, but it is reasonably clear that whatever measures are finally adopted will result in requirements for new and/or more elaborate installations. As a result, many knowledgeable people think that there is likely to be a major increase, possibly a doubling, in the Air Force's military construction program in the next few years to provide alert, protected, and dispersed facilities for both missiles and aircraft, for both SAC and ADC.

A high proportion of these new installations will be of novel design, sometimes embodying new or esoteric principles. If the designer of these installations is to do a good or even a barely adequate job, he is likely to find his reserves of experience, knowledge, ingenuity, and technique strained to the limits. In all likelihood he will find himself "going to school" again and working with strange and sometimes difficult collaborators.

Today we are going to concentrate on a very special kind of protective construction, the deep underground building. While some of my remarks will be directed toward exposing the military role of such buildings, most of what I have to say will be on the general nature and likelihood of thermonuclear war, and on the quality of the preparations needed to meet the war threat. Compared with the severely technical problems to which most of the other papers are addressed, these last two subjects may seem too philosophical or policy-oriented to be appropriate for this audience.

Most of the people here are either designers, technical advisors, or members of staffs of service and support organizations. Policy is not supposed to be made at this level. (Most of this audience would probably claim, "We just receive orders and carry them out.")

In practice, of course, it is not like that. No matter what the organization chart may say, much policy is made at the designer's level. His proposals filter upward and are often changed in the process of filtering, but usually most of the major decisions have already been committed by the designer before the filtering process has even started. It is not formally or legally so, but it is so.

Technicians often like to think that all they do is explore the various alternatives available to the policy maker and describe these alternatives in some appropriate level of detail; then the policy maker chooses which alternative he likes best. However, it is much too difficult and time-consuming for the average staff, or even engineering firm, to explore many alternatives. After a preliminary (often, very preliminary) examination, the staff will generally decide on one or two alternatives as the right ones to explore, and then will present these alternatives to the policy people to either accept or reject. The choice of which alternatives are worth exploring will be based partly on the guidance available from the top, partly on the conscious and subconscious view the "technical" explorers have of what is important and what is not, and partly on the background and experience of the designers.

The trouble with this way of choosing which alternatives to explore is that the guidance from the top turns out to be circular, having been strongly influenced by the views of the people being guided. What is important and unimportant is judged more in terms of the day-to-day peacetime and bureaucratic context than by performance during a hypothetical war of the future. Therefore, only solutions which seem "reasonable" and "safe" are considered, where "reasonable" and "safe" are, naturally, measured by peacetime, not wartime, standards. Finally, and most important: in a rapidly changing technology, the background and experience of the most readily available designers may not be even remotely appropriate.

Because many of the most important decisions may be made by default rather than by conscious choice, few if any of the people involved will even realize that a decision has been made. In addition, because the "official" policy makers are both busy and lack technical background, the presentations made to them tend to be short and superficial. Therefore, a lot of pretty important decisions also inevitably get made in deciding what packages to present for acceptance or rejection.

For all these reasons we think it is worthwhile to take a small portion of the program to discuss the basic "why and wherefore" of protective construction policy to a predominantly technical audience.

Let us start by discussing the source of most of our difficulties, the rapidity with which the art and technology of war change. Every five years seems to bring a revolution which introduces tremendous difficulties for both the planner and designer. Lessons and techniques learned at bitter cost may be worthless before they are fully mastered.

Today it takes about four or five years from the time we first lay down the requirements for a building until the Beneficial Occupancy Date. Most people think that a building ought to have a useful life of at least twenty years. This is much too much to expect of a military structure in the world of today. However, such buildings should not be planned with the expectation of lasting less than five or maybe ten years. This means, if my arithmetic is correct, that we should today be considering the 1965-1970 time period and if we want much more than a 5-year life for our buildings, or even a little more time for thinking and research, we should be looking at the 1970-1975 period also. To my knowledge this kind of forward thinking has not been customary in the protective structure field.

If this were a classified session I could illustrate this point by giving the history of several weapons systems which became prematurely obsolete--in some cases before they had been fully procured--because the designers or planners showed insufficient foresight or imagination. Given the rapid changes in the art of war, it is inevitable that this should happen occasionally, but responsible planners should try to guard against this possibility. In all too many cases planners or designers have failed to anticipate even a mild degree of technological progress. In some cases, "crash" programs designed to provide a quick fix have also been under-designed, so that even they risk being obsolete before they are fully installed or procured.

Under these circumstances there is a heavy responsibility upon engineers and designers to work closely and sympathetically with the professional military planners and operators so that a full exploration of all the possibilities can be made. It is especially important to build systems which can be designed either to meet the full range of threats, or that are capable of being retrofitted or improved to meet increases or changes in the threat.

This means that we have to look for new ideas and gadgets, push research and development, and take some technological gambles. It seems to be true in almost every other field of military technology that the majority of really new ideas did not originate in official requirements, or in massive projects pushed by the official agencies or large companies, but rather came from the small percentage of successful ideas pushed by individuals or small outfits (some of them almost fanatic in character). What I am saying is that if you look at the large number of ideas which are generated by small outfits and individuals, very few of them are successful, but it is these few which are the basis of much of our military systems today.

In the field of protective construction, however, I think it is fair to say that the profession has lagged significantly behind the need. It has not been pushing the state of the art. On the whole it has waited for leadership from the government. People were talking of the design of structures suitable against high explosives when they were faced with the 20-KT bomb; by the time they had allowed for the 20-KT bomb, they were facing the megaton bomb carried by a bomber, and today, of course, they face the ICBM. People are designing buildings for protection against single bomb drops, but the enemy may have thousands of ICBM's in the 1965-1975 time period; people are designing buildings to survive inaccurate low-yield weapons, even though it is almost certain that the enemy will improve his accuracy and yield if he finds it necessary to do this to destroy an important target.

Possibly even more difficult than the intellectual problem of planning for an uncertain world is the related psychological problem of taking seriously the performance of these buildings in a possible future war. Many people seem to believe, or act as if they believe, that preparation for war is a sort of Alice-in-Wonderland activity unrelated to any possible real eventuality. I recently had occasion to give three 2-hour lectures at Princeton University on the reality and likelihood of thermonuclear war,* the threat thereof, and the need for being able to fight a war as well as being prepared to deter one. I would like to spend my time going over these lectures, rather briefly of course, and try to point up how they relate to our problems.

The first talk was entitled, "The Nature and Feasibility of Thermonuclear War." It tried to answer the questions, "Is thermonuclear war a

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*Three Lectures on Thermonuclear War, Princeton University Press, to be published Spring 1960.

practical activity?" Can it be an "extension of policy by other means," as Clausewitz said? Most people, including the experts, don't think so. They seem to think that only an insane man would initiate an all-out thermonuclear war. When they talk about war, they talk about "mutual annihilation," "inescapable end of civilization," "balance of terror," "thermonuclear stalemate," "destruction of all life," etc. The analogies that are used--"two scorpions in a bottle;" "two people on a single keg of dynamite, each with his own match;" "two heads on a single chopping block;" etc., reinforce this apocalyptic view. The most optimistic phrase I have seen used is, "a war in which the survivors will envy the dead." The reason it is optimistic is because it explicitly concedes that there will be survivors.

All of the above give the implication that war is not an activity in which rational human beings can engage, since presumably no democratic and few totalitarian nations would commit suicide in the pursuit of some objective. There just is no legitimate objective of public policy which enables one to say, "Let's end it all for everyone."

In some ways this view of war is comforting. The reason why the "balance of terror" theory is comforting is obvious. If a war is this disastrous--mutually disastrous--surely no one would start one. Of course, you have to worry about accidents, you have to worry about miscalculations, and you have to worry about crazy people--but that is all. And in an issue this big, it is hard to believe that the accident or the miscalculation or the crazy man is to be seriously considered. In any case even if the war occurs for one of the above irrational reasons the decisions maker still cannot be blamed for any lack of preparation to alleviate the consequences. Annihilation is annihilation.

Some of my colleagues and I have a quite different view of war.* We agree that it is terrible. We agree that it would be an unprecedented catastrophe--but not necessarily an unlimited catastrophe--and most important of all, not necessarily a mutual catastrophe. The balance of terror is not automatic. It is very possible for one side to out-compete the other to such a degree that the superior side could safely start a war and get away--maybe scot free, or maybe with appreciable damage to himself, but not so much damage that he would prefer not attacking to attacking. I will discuss in a moment exactly what I mean by this remark, but before I do I would like to ask, "How much damage from retaliation might a potential aggressor such as the Soviet Union expect to receive?"

Before you can answer a question like this, you must have the answer to two questions: "How many bombs can the U.S. and its allies drop on the USSR after it strikes, and then we strike back with a damaged and uncoordinated force in the teeth of an alerted air defense and against a warned and possibly evacuated civilian population?" "How much damage will these thermonuclear bombs do to him?"

The first question has been discussed by Albert Wohlstetter in an article in the January 1959 issue of Foreign Affairs called, "The Delicate Balance of Terror." He concludes that it is difficult to guarantee even a small retaliation, and very difficult to guarantee what might be called a moderate or large retaliation (some hundreds of bombs dropped on target). It is also important to realize that even a quite large retaliation probably

*Kahn, Herman, Major Implications of a Current Non-Military Defense Study, The RAND Corporation, Paper P-1497-RC, November 7, 1958. Also published under the title, "How Many Can Be Saved?", Bulletin of the Atomic Scientists, January 1959.

does not annihilate a nation such as the Soviet Union. The Soviets might easily end up suffering less damage than they suffered in World War II, from which they have recovered rather well.

All in all, it seems fair to say that a sober examination of the probable course of a war indicates it is rather unlikely that a moderately competent aggressor who has prepared to accept his victim's retaliatory blow would be set back more than 10 or 15 years in his development--possibly much less. In arriving at this conclusion we examined eight phases of the war and postwar period:

1. Various phased prewar military and civil defense programs
2. Wartime performance of these prewar programs under different attacks
3. Magnitude of the fallout problem during the first few days
4. Survival and patch-up
5. Maintenance of economic momentum
6. Recuperation of the economy
7. Long-term medical problems
8. Genetic problems

Our conclusions on the "toughness" of modern nations such as the Soviet Union or the United States are given in Report on a Study of Non-Military Defense, The RAND Corporation, Report R-322-RC, July 1, 1958. This report discusses systematically, if briefly, what is involved in surviving a rather massive nuclear attack and restoring a semblance of prewar society in about ten years or less. In this talk I would like to touch on only phases 6, 7, and 8 above, since it is these phases which seem to many people to present the worst problems. Let us discuss them in reverse order.

Most of you are probably aware that many biologists and geneticists are worried about the genetic effect of peacetime testing; some even talk as if the future of the human race is being jeopardized by exploding a few bombs a year in the Pacific Ocean or the Soviet Arctic. Now, if you believe this (and some pretty competent people seem to either believe it or come close to believing it), then you immediately ask yourself, "Well, what would happen if you exploded a lot of bombs inside a country rather than a few far away?" Surely, if the one is dangerous, the other is cataclysmic. Well, the one may be dangerous, the other more dangerous--but not cataclysmic!

In one hypothetical Soviet attack against the U.S. which we looked at, it turned out that the survivors of the war received an average effective dose of about 200 roentgens of radiation to their reproductive organs. This is an enormous amount of radiation. It is about one or two thousand times the amount we expect people in the U.S. to get as a by-product of the test program. It is about 50 times the amount they would normally get from natural sources. It is a large and frightening dose, but it is not an annihilating catastrophe.

If our current ideas are correct, the genetic effects of this amount of radiation might cause 1 per cent of the children to be born seriously defective who would not otherwise have been defective. While this is a large penalty to pay for a war (and more horrible still, one might have to continue to pay a similar though smaller price for twenty or thirty or forty generations), it is still very far from annihilation. A woman came up to me once and said, "I don't want to live in <u>your</u> world where 1 per cent of the children are born deformed." The answering remark which I made, while heated, was I hope justified. I said, "Lady, it is not <u>my</u> world, I am only describing it. There are about 4 per cent born defective today, so if you mean what you say, you can only commit suicide." The point I am making is that a war is a very horrible thing--but so is peace in a way. War is just more horrible. It's like that. And this in turn means that this extra horribleness of the war may or may not deter a nation from going to war.

The experts talk about other medical problems than the genetic ones--for example the bone cancers and leukemias which might be caused by Strontium-90, and the other life-shortening effects of radiation. They sometimes seem to be saying that these are so horrible that no nation would deliberately start a war. Here again, examination indicates that while the problems are horrible, they may well be within the limits we are accustomed to in our normal peacetime activities. For example, it could easily be true, as some scientists have claimed, that every time a large-yield bomb is tested in the Pacific Ocean or in the Soviet Arctic, thousands of people who are currently alive will get bone cancer or leukemia. This number may also turn out to be zero. Nobody really knows, but respectable estimates running into the thousands have been made by some scientists. (I mean by "respectable" that the scientist making the estimate would be willing to publish it in a technical journal and would not be considered guilty of unprofessional behavior.)

Again, this automatically makes many people say, "Well, if a few bombs in the distant Pacific or Arctic cause this much trouble, wouldn't a larger number of bombs closer in be totally catastrophic?" Some military experts even refer to the so-called backlash effect of an attacker's bombs as being an automatic deterrent. Would that the problem of deterrence could be solved so easily.

As before, more and closer bombs will cause more trouble than fewer and farther ones, but not necessarily that much more. In typical wars which we have looked at, in which people have taken moderate protection--have done things which the Russians, for example, seem to be doing or thinking of doing today--the effects of fallout can be reduced drastically. With such preparations and some advance warning (the more preparation the less warning is needed) most people can survive the fallout effects. The war might result in an average life shortening of less than one or two years in some situations and five or ten years in others. But in any case, life will go on. War would not necessarily involve annihilation of civilization. The above estimates apply when you get hit very hard--when the country accepts hundreds of bombs, more or less on target.

We have also looked at the possibilities of economic recuperation, and we talk in the following way: Most people--laymen, and some experts--look at the highly integrated character of a modern economy and argue that a nation is like a body; destroy the heart or other vital organ, and the body dies; a few cells may linger on for a while, but they will die very soon.

We don't think it is like that. We looked at the U.S. and we divided it into two countries--an "A" country with the largest 50 to 100 cities and a "B" country--the rest of the United States. Now the A country cannot survive without the B country, but the B country, as far as we can see, can survive without the A country, and it also has the resources and skills needed to rebuild the A country in, say, ten years. In other words, we shouldn't think of a country as being analogous to a body, but rather think of the major urban areas, the A country, as trading with the B country, the rural areas, towns and small cities. Now, if England were destroyed it would not shock most people to find that we could get along without that country even though we had previously had a very large and valuable foreign trade with England. The A and B country relationship in the U.S. seems to be like that.

Further, it is possible, by using existing construction and by improvising fallout protection, to prepare the B country to receive evacuees from the A country and protect them reasonably satisfactorily. In the summertime, it would probably be no problem for the Soviets to make this kind of shift on days or hours notice. In winter they might need more time. Incidentally, current (1958) Russian civil defense manuals indicate that they are making such preparations. In addition the Russians claim to have given, last year, to every adult in Russia a 22-hour course in civil defense, with a compulsory examination at the end of it so that they can check if every citizen knows what he is supposed to do.

Such an evacuation is very plausible in any kind of tense situation in which the Soviets are trying to force us to back down or to negotiate from strength. They can put a great pressure on us by evacuating their cities. While this gives us warning, we may not act on the warning. The Soviets may only be trying to convince us that they intend to go to war unless we back down. Just because they are trying to scare us, we may resolutely refuse to be "bluffed." In any case, as I will try to make clear later, the only practicable counteraction we may have might be to evacuate our own cities--an action for which we have made almost no realistic preparations.

There are about 50 million Russians living in the largest 135 Soviet cities. If, say, 80 per cent of these 50 million evacuated to the Soviet B country and 20 per cent stayed behind to operate the cities, the Russians would find it possible to carry on all important functions, and yet leave only about 10 million citizens at risk in these cities. Furthermore, having evacuated the bulk of the population, it would be possible to evacuate the rest very expeditiously. As long as our ICBM force is small, they would have plenty of time to do this before our retaliatory bomber force would reach the bulk of these cities.

Under those circumstances, if the Russians struck us first with a reasonably successful strike, and we struck back with a damaged force, unless things go almost incredibly badly for the Russians, U.S. retaliation could not kill more than five or ten million Russians--probably much less; that is, the Russians might lose only a fraction as many people as they lost in World War II. There may well be circumstances where they might be willing to face this prospect and go to war.

Now, why is this--the fact that there are circumstances under which the Soviets might find it practical to go to war--important to technical people like those in this audience? It is important because today even designers of military buildings sometimes do not take war seriously. Jack O'Sullivan is going to give a talk following mine in which he will contrast the military performance of the structure as opposed to the peacetime performance of the structure, but let me make a comment on this subject.

If one puts up a building in peacetime and it operates reasonably well, the fact that it won't perform in wartime may never be noticed. At the most, it is only noticed once. On most nights when you go to sleep, the building is still there when you wake up the next day. The fact that there are some inadequacies in the design of the building from the viewpoint of wartime performance may not be as apparent to the users as some much more trivial inadequacy that shows up in the day-to-day performance. If, for example, there are not enough toilets, everybody notices and complains about it.

Quite different things are important in a war, of course, but you are not normally fighting a war. For this reason, there is an almost overwhelming tendency to compromise the design, particularly where there is uncertainty, in favor of better peacetime performance or lower costs. The wartime performance is neglected. No designer would take one chance out of ten of his building falling down in peacetime, but he will take much bigger chances that the building will prove to be a total failure in wartime. (Total failure is defined as the enemy's being able to destroy the building at trivial cost to himself.)

I am trying to make a war seem more real in order to make the requirements of living in a war also seem like something to be worried about, rather than academic and hypothetical.

The second lecture I gave at Princeton was called, "The Formulation and Testing of Strategic Objectives and Plans." It discussed "why" and "how" the United States or the Soviet Union might fight such a war. Let us just discuss "why" here.

As I said, it is very hard to convince most people that either the United States or the Soviet Union would ever deliberately initiate a conflict. In the Princeton talks, in order to make it plausible that this could happen, I used a debating trick of a slightly unfair sort. Let me use this same debating trick again. You may be familiar with the story attributed to Bernard Shaw in which a young man walks up to a very attractive young lady and says, "Will you sleep with me for a million pounds?" She thinks about it, and hems and haws. (A million pounds is a lot of money, worth five million 100-cent dollars when Shaw wrote the story.) And she finally says, "Well, all right." The young man then asks, "Will you sleep with me for two pounds?" She slaps his face indignantly and says, "What do you think I am?" He says, "Well, we have established the principle. Now we are just haggling over the price."

I want to establish a principle and then haggle over the price. Let me start by asking whether the United States would ever initiate a thermonuclear exchange. We usually say no, but we also say that the Russians can't do certain things in Europe. We have commitments to NATO, for example, and if the Russians attacked West Germany, we wouldn't stand idly by; we would attack the Russians--or would we? Well, that is related a little bit to the price you have to pay.

Now, let me make a flat prediction: no President of the United States would, if he had to wait two or three days to cool off, initiate a strategic action if he and his advisors were convinced that the Russian retaliation would kill 175 million Americans; that is, if everybody in the U.S. would be killed. I believe that this statement is true no matter what the Russians did in Europe.

Now, at the Princeton lectures there were a fairly senior group of people in attendance, many being either policy makers or their advisors. Most of them just couldn't visualize the United States failing to honor its commitments. I therefore asked anyone in the audience to stand up if he thought the United States would honor its commitment to any nation, if honoring those commitments meant the death of every single American. Nobody stood up. Everybody either thought it was just too bizarre a position to take publicly or was too polite to try to embarrass me. However, it is quite probable that even those who thought they would react realized that their position was bizarre. Actually, everybody would probably react if they had to act in anger. But, if you have to think about it, you just can't do it.

Well, then, one has to ask, "What's the price?" I have haggled over the price with many people, at least hypothetically. I have done this with both Europeans and Americans. The prices we negotiate run between two million and sixty million dead--that is, they think that the U.S. would be willing to loose this many people in punishing an aggressor. (This last figure is obtained by taking one-third of the population--just barely less than half.) This is the range that people estimate. I don't know what the price really is, but even these crude estimates tell you something.

If you don't have faith that your military establishment can keep the casualties down to, say, less than fifty million dead and in addition preserve enough of the country and prewar environment so that "the survivors will not envy the dead," then Khrushchev can bluff you and win the bluff. Furthermore, Khrushchev can do this bluffing safely because he can do it the way Hitler did it. He can test your resolve experimentally. He can raise the U.S. to a peak of tension, drop the tension, raise it, drop it, and so find out exactly where the U.S. stands. It is important to realize that he can do this testing relatively safely.

Let me now ask the other side of the question. Are there any circumstances in which the Russians would attack the United States--attack deliberately and with malice aforethought? Well, that too depends on the price and the alternatives. It is probably true today that no reasonable decision maker would start a war for the sake of what might be called positive goals. That is, even though the intelligence briefing may start out with the remark that the Russians seek world domination, Khrushchev, in fact, probably would not start a war to achieve this goal. He may want it, and want it very much, but hopefully not enough to start a war in cold blood--at least not unless we have been incredibly careless.

If he starts a war, he probably starts it because the risks of not going to war loom larger in his eyes. Let me give you an example of such a possibility which seems quite realistic to me. In 1956, the Hungarians had a revolution which the Russians suppressed with much bloodshed. There was a lot of pressure on the United States to intervene in that revolution. We didn't. In fact, there are reports that we did exactly the opposite--that we broadcast to the Poles and the East Germans not to rock the boat and that no American aid was on the way. Let's assume that we had given in to that pressure and intervened. The Russians would then have been faced with three fairly serious choices:

(1) They could do nothing. This would almost automatically mean a Polish and East German revolt. (They almost revolted, you remember, without any hopes of American aid.) Such a revolt would mean serious repercussions within Russia. We don't know how serious these repercussions would be, but we know that Russians worry about political stability; they might think of it as intolerably serious.

(2) Secondly, they could fight a limited action. But that too would be quite risky. If the U.S. fights a limited action and sticks with conventional high-explosive weapons, we are going to lose, just by the sheer weight of numbers. If we go to atomic weapons at a high enough level to win, it is a little hard for some people to believe that the war will stay limited. At least, it shouldn't surprise us if the Russians view the prospect of a limited atomic war close to their heartland with serious misgivings. They might easily believe that we Americans were quite capable of suddenly expanding the scope of the war with a surprise attack at their strategic forces. They might therefore argue that their third alternative was safer.

(3) The third possibility is this: rather than wait for the satellite revolt or the limited war to erupt into a general war at a time chosen by

the Americans, the Russians might decide to hit us right away. They could argue that this would guarantee to them the all-important first strike--at least if they hurried.

The third lecture I gave at Princeton was called "World War I through World War VIII." In this lecture I discussed the characteristics of eight real and hypothetical wars, each one a technological revolution ahead of its predecessor. These wars are assumed to occur as follows: 1914, 1939, 1951, 1956, 1961, 1965, 1969, and 1973. You will note that while there are five years between wars in the middle period, there are only four years between wars in the later period. This was deliberate. Both we and the Russians seem to be learning how to do research and development better, and we are both spending more on it, so we can probably expect things to go faster in the future than they have in the past. I have already discussed the many problems that a designer faces in trying to keep up with technological changes. Let me therefore skip that point and discuss the question of gaps between two military powers. There are three types of gaps one might worry about:

(1) The Research and Development gap, where the other side knows more about some weapons system or weapons effects than you do. This is a serious gap, but it is not disastrous, at least not for that year. It may mean a dim prognosis for the future, of course.

(2) The procurement gap, where the enemy has bought more of some weapons systems than you have. Whether this kind of gap is serious or not depends on what you are trying to do. It is almost always bad for national prestige, however.

(3) The operational gap, where the enemy has, or can present you with, a threat which you cannot solve satisfactorily. Typical examples would be:

German use of submarines and poison gas in World War I; in World War II the German break-through at the Ardennes Forest; and the Japanese attack on Pearl Harbor and Singapore.

Clearly the three kinds of gaps can be independent. It is perfectly possible for the enemy to have a much larger force than you have, and for you still not to have an operational gap. This is, in fact, the popular view embodied in the "minimum deterrence" theory, which holds that as long as you have enough armed forces to destroy the enemy's cities, it is irrelevant to you how much armed forces he has. While there is a germ of truth in this theory, as it is ordinarily applied, it can be very misleading, as I will discuss below.

In the Princeton lectures I tried to show almost on a year-by-year basis, starting in the early 1950's and ending with the early 1970's, that for the first years of that period at least, it has been historically true that certain kinds of gaps have developed in our posture. What is even more frightening, these gaps are likely to increase both in number and magnitude over the next few years.

Let us consider now the general kinds of gaps which could develop, with particular emphasis on the role of the most important kind of underground building, which is the headquarters. There are at least three basic situations and corresponding gaps which one can distinguish. All three can be phrased (perhaps a little misleadingly) in terms of deterrence. The situations are:

(1) deterrence against direct attack

(2) deterrence against extremely provocative behavior

(3) deterrence against a continuous level of moderate provocation

The most relevant calculation for the Type 1 deterrence situation would envisage a surprise attack by the enemy, and we would estimate the amount of damage inflicted on our strategic forces in this surprise attack. We then launch a retaliatory blow with our own damaged forces, where the timing and coordination is determined by the enemy's initiative, and where our force must penetrate the enemy's alerted air defenses. Finally, we estimate the damage done to the enemy by that part of our force which has penetrated the enemy's defenses, giving the enemy full credit for any civil defense preparations he has made.

We have already pointed out that under these circumstances we may do substantially less damage than we would have done if the enemy had not struck us and if we had been able to choose the time of the attack. One of the troubles with the "minimum deterrence" theories that I referred to previously is that when its effectiveness is calculated, the factors I have just mentioned are not generally included. As a result, people sometimes get optimistic ideas of how effective such minimum deterrence forces are going to be. They also tend to evaluate the effectiveness of the deterrent force when it is not being strained; but as we just pointed out in our discussion of the Hungarian crisis, it is very plausible to imagine circumstances in which even a seemingly adequate deterrence force is strained.

The main functions of our underground headquarters are:

(1) to recognize that there is an attack;

(2) to assess the damage;

(3) to make the decision as to what to do;

(4) to communicate these decisions to the proper places;

(5) to regulate and monitor the ensuing activities.

These are all very critical functions now, and are likely to be even more critical in the future.

One reason why the decision and communication problems may be important is that everybody--even those who find the idea of deliberate war incredible--is likely to be very worried about the possibility of starting a war by accident or miscalculation. Because there will be so many people with authority over weapons systems, it will be necessary to make it clear to everyone that they cannot act on their own, no matter how many fireballs or mushrooms they think they see. Unless they receive an official, verified order, they will not be allowed to press the button, irrespective of any evidence they think they have that the war has started. If this were not done, one could almost guarantee that a war would occur by accident.

This, in turn, means that there is a potential weak spot in the system if the enemy can destroy your headquarters. He can then delay your forces from reacting. This gives him two possible advantages. First, it gives him more time to destroy these forces; and second, and probably most important, it enables him to use post-attack blackmail in attempting to disarm these forces. That is, he can carefully avoid your cities in his attack and then try to prevent an attack on his cities by threatening to take out your cities, say 5 for 1. If he has been reasonably effective in his first attack, this is a very credible threat, and if he makes it dramatic enough it might influence your actions. For example, he can say that if you destroy Moscow, he will destroy New York, Philadelphia, Washington, Chicago and Los Angeles. If you take out Leningrad, he will take out San Francisco, Detroit, Pittsburgh, Boston and New Orleans, etc.

He can finish his list with a remark to the general effect that "You know better than I do what kind of country you want to have after this war is over. Choose which cities you wish destroyed."

Regulating and monitoring other activities is also important. For example, insofar as a retaliatory force is composed of manned bombers, we don't want our own defenses to fire on these manned bombers as they pass out of the United States. The flight plan and identification data needed to prevent this probably should be kept in some centralized headquarters. Another monitoring and control function is that of adjusting target assignments for our forces to make up for the damage that he has done to us. We will then need some sort of centralized information gathering, data processing, and assessment facilities. If the enemy can destroy the buildings that house these facilities, he will degrade our capability.

The major threat against our headquarters buildings, as far as Type 1 deterrence is concerned, is the ICBM, possibly supplemented by submarine-launched missiles or even bombers. It is important in laying down the requirements for these buildings to realize that if the buildings have not been put very deep underground, the enemy may be able to design special missiles which carry unusually large warheads for the specific objective of destroying these buildings. That is, when you put these buildings underground you challenge the enemy to destroy them, and you shouldn't be surprised if he reacts to that challenge, or at least tries to.

Let us go on now to Type 2 deterrence--deterring the enemy from provocative behavior. In this case the requirements for our buildings may be much more severe. We must now try to make it credible to the enemy, to our allies, and to ourselves that if he is sufficiently provocative, we are willing to initiate a thermonuclear exchange and strike him, and then accept his counterstrike. This means that we must be able to terminate a war on terms which are satisfactory to us and our allies. Some of our headquarters installations at least must be able, then, to survive not only the enemy's missiles but even his manned bombers. Unless we have some such installations operating on the second or third day of the war, it is unlikely that we will be able to terminate the war under any reasonable conditions. It is rather interesting to note that in this particular case we may have no research gap and no procurement gap in the sense that we have more and better equipment than the enemy does, and yet there may be a very large operational gap. We could even have a superb offense force; yet if our air defense and civil defense are inadequate for accepting the enemy's retaliatory blows, we have an operational gap.

Sometimes parochial people measure our offensive power, our power to regulate enemy behavior, just in terms of the number, quality, and operational capability of the offensive vehicles we own. But actually our ability to regulate the enemy's behavior depends as much, if not more, on our active and passive defensive capability as on our offensive capabilities. Unless the enemy is incredibly careless, we cannot hope to destroy an overwhelming majority of his force on the ground; therefore, unless we can alleviate or accept the retaliatory blow launched by his surviving SAC, we may be deterred from trying to regulate his behavior.

If you want headquarters buildings to survive an attack by manned bombers, they may have to be much deeper and much stronger than if they are just to survive ICBM's. A manned bomber can carry a very large bomb and deliver it very accurately. It is of course possible that you will try to protect these buildings with a certain amount of active defense against the bombers if the buildings themselves are intrinsically very tough.

Let us now consider Type 3 deterrence--deterrence against a continuous level of every provocation. It is clear that Type 3 deterrence cannot be expected to work 100 per cent of the time. The enemy will do at least moderately provocative things occasionally, and you will have to either acquiesce or react against them. However, there is a special type of capability you can have which may regulate the provocateur even more than the military threats we mentioned previously. This is an explicit capability for greatly increasing your strength very rapidly. For example, in June 1950 the United States was engaged in a great argument as to whether our military budget should be 14, 15 or 16 billion dollars. Along came Korea and Congress authorized 60 billion dollars--a fourfold increase.

No matter what the Russians did in Korea, whether they won or lost, that authorization represents an enormous military defeat for them. However, it was almost 2 or 3 years before the authorization was translated into increased budgets and increased military power. It is very valuable to have a capability to increase our expenditures, but this capability becomes even more valuable if it can be done in a year or so. If the Russians know that if they allow international relations to deteriorate, we will probably go into an effective crash program, then they may be much less willing to let international relations deteriorate.

Some members of this audience may be surprised to learn that many economists believe that we could, for a short time, spend about 200 billion dollars a year on defense without undue hardship. This probably represents less than 40 per cent of what our GNP would be under mobilization conditions, and I might mention that in World War II, at the peak, we spent 45 per cent of our GNP on defense. And I think most of you realize that on the whole, the population suffered little hardship. Such levels of expenditures would of course mean the temporary termination of non-military construction and production of consumer durable goods, but if the safety of the nation were clearly at stake, I think we would be willing to do these things. The problem is, of course, "Would we have the time to put in a useful program?" It takes a long while before you can spend these large amounts of money. You cannot build an adequate military system overnight. In particular you must have a basic military establishment of the proper sort if you expect to be able to expand it within a year or so to the point where you are prepared to "fight" a war in addition to being able to deter one.

I think it must be clear to everybody in this group that the various central headquarters are among the most basic components of any expanded system designed to "fight and terminate" a war rather than "deter one." What I am saying is that when you are building a military system you should not use the one-horse-shay philosophy and build all components to an equal level of vulnerability. It is of course very bad to have weak spots. The enemy may exploit them. However, it is not tragic to have a few strong spots--particularly if there is any possibility that the rest of the system may be brought up to the same level.

Therefore, since these buildings should last 5 to 10 years, their vulnerability should not necessarily be equivalent to the most vulnerable part of the rest of the current system but equal, possibly after retrofitting, to the basic vulnerability you may possibly want during the whole time period when these buildings will be used. The possibility of the 200-billion-dollar crash program I have just described should be considered.

I would like now to make some additional comments on the vulnerability of different kinds of headquarters installations. The following table shows some of the weapons effects to be expected from ground-bursting a 1-megaton and a 100-megaton thermonuclear weapon.

While the range from 1 to 100 megatons is a reasonably large one, it may not cover the entire region of interest. It is very possible that Russian bombers could carry air-to-ground missiles with less than 1-megaton warheads, and in any case published reports have indicated that the American Minuteman and Polaris systems may, at least initially, carry warheads smaller than this size. On the other hand, while the idea of a 100-megaton weapon may strike most of this audience as rather extreme, it is not at all unreasonable to design against the possibility that such bombs may be developed and carried in the 1965-1975 time period. In fact, 100 megatons may not exhaust the upper range of the possibilities or at least speculations. There have been several published references to the possibility of 1000-megaton or begaton bombs (sometimes more accurately if pedantically referred to as gigaton bombs).

If we do not harden any of our installations, it might be very reasonable to suppose that the enemy will not bother to develop very large bombs. But once we have built hardened structures of the sort we are discussing here, then we have challenged him, and if he can meet this challenge by building 100-megaton bombs and the corresponding delivery systems, then he is very likely to do it. Enemies are like that; always spoiling and obsoleting a perfectly good design. There are restrictions on the maximum size bomb that can be delivered practically, but very few technical people would bet that these limits will be much below 100 megatons in the 1965-1975 time period.

The table gives the estimated weapons effects. Let us now look at the resistance of some typical designs. The normal reinforced concrete building

SURFACE BURST CHARACTERISTICS

Peak Over-pressure (psi)	Distance of Burst in Feet		Remarks
	1 MT	100 MT	
35,000	500	2300	Crater Radius (Rock)
14,000	650	3000	Crater Radius (Soil)
8,800	750	3500	Rupture Zone (Rock)
4,000	1000	4600	Rupture Zone (Soil) and Lip (Rock)?
1,600	1300	6000	Lip (Soil)?
1,000	1500	7000	
500	2000	9000	
200	2700	12,700	
100	3500	16,400	Fireball Limit
25	6300	29,000	
10	9800	45,000	
2	28,000	130,000	
	120	400	Crater Depth (Rock)
	150	500	Crater Depth (Soil)
	150-200	500-600	Permanent Deformation (Soil)?
	200-400	600-1200	Permanent Displacement (Rock)?

that one builds in this country would probably accept 2- to 5-psi overpressure without being completely destroyed. However, the equipment inside may become inoperative at less than 1 psi because the typical building has weak spots such as doors which can be blown in, allowing the equipment inside to be damaged.

Such a building is not even worth considering for a headquarters intended to survive attack. The next alternative is a blast-resistant aboveground structure. About the most resistant building one would design here would be for 25 psi. This might be all right for the immediate present, but even a few years is sure to make such designs obsolete.

We can then go to flush basement-type structures. The practical economic limit here would be about 200 or 300 psi. For greater blast resistance one would want to go truly underground. I would judge even a few hundred psi to be very marginal, but some people disagree with me.

The next structure might be fifty to two or three hundred feet underground. Except possibly for ground-shock problems, it is very likely that this kind of structure would survive hundreds and maybe even a few thousand psi, depending on the design of the cavities and tunnels and on the quality of the rock.

The next structure might be 300 to 1000 feet underground. This would probably be in the thousands of psi range, and even the ground-shock problem should be reduced.

Finally, there is the really deep (about 2000 feet) underground structure. This should not even have ground-shock problems in the less than 1000-psi range, and it should be able to withstand almost direct hits of small (less than 1-MT) bombs and near misses of larger bombs.

All of the above vulnerability statements are just conjectures, but I have some confidence that analyses we have currently underway will corroborate most of the remarks I have made.

We mentioned before that if one does not want to go all the way in reducing the vulnerability of headquarters structures, one might build in a capability for retrofitting. One way this might work in practice would be that instead of building a 100- to 200-psi subsurface basement-type structure, one might build the basic structure in tunnels hundreds or even thousands of feet underground. Everything else, such as doors, ventilators, communications, etc., would be built to a 100- to 200-psi level. If later on one decided one needed increased protection, one could retrofit by changing these doors, adding additional harder communications, and even getting rid of the ventilators completely by using reactors for power. It should be noted that a building of this sort can still operate and perform its function, even if all of the entrances have been caved in, if it can still communicate with the outside world. Since communications can be made extraordinarily hard, it is essentially possible to make this building almost completely independent of the surface.

I would like now to make a few comments on the role of theoretical calculations in the design of such a structure. If one has to choose between theoretical calculations and advice from competent people with relevant experience, there is no doubt that no matter how good the theoreticians are, one would almost always prefer to rely on experience. But, nobody today has experience in the structural problems raised by thermonuclear weapons. Very few engineers, even including those who have worked on protective construction, are familiar with the known weapons effects, particularly in the case of the deep underground building. This means that one has to be careful in applying peacetime experience to a building whose main usefulness is during a war. We are always having arguments with unsympathetic people who consider peacetime considerations overriding. The dominating consideration must be, "Does this structure do the wartime job it was designed to do?"

There are many kinds of calculations people can make. Some of them are extremely reliable because they depend on such trustworthy things as Newton's laws, conservation of mass, conservation of energy, etc. Others are reliable in the sense that they can indicate when a structure will fail, but they cannot say when it will survive. Sometimes the situation is the other way: one can say when something will survive such and such a condition but cannot say accurately when it will fail.

Let me give some examples. It is quite likely, for instance, that the calculations that H. L. Brode of RAND has done on the transmission of shock in tunnels can be done in such a way as to overestimate the effect of the shock. Every loss he takes account of, undoubtedly does occur. However, it would be difficult to get a close underestimate of the same thing because there may be loss mechanisms which were not taken into account and which we do not know about.

Therefore, Brode can make a calculation on the basis of which he can state with some reliability that a door should survive at a certain place in the tunnel. When he says it will fail, he cannot be nearly as certain.

By contrast, Paul Weidlinger's calculations on the failure of tunnels are likely to have the other trouble. When he calculates that a tunnel fails by a certain mechanism, it very likely does; but he cannot be sure that the tunnel will not fail by some mechanism which he does not understand or has not allowed for.

It is this last type of calculation which is most troublesome. It can tell you when you are in trouble, but not when you are safe. All that you can do is to be as careful as you can, give full consideration to the phenomena involved, put in some insurance by over-designing (at least when it is cheap), and then hope for the best. We would like to do more but we simply cannot—at least until we run full-scale tests. All we can do is just face the fact that to some extent the working of our installation depends upon faith. However, as Jack O'Sullivan will point out when he discusses the ground-shock problem, "One can often by suitable design run away from the problem." Wherever possible, this is a preferred solution.

Let me finish by telling a little story which tries to illustrate that the subject we are considering today may, in the future, seem as glamorous as that of high-speed aerodynamics or missiles. About four years ago the Scientific Advisory Board to the Chief of Staff had what they called "a think session" in Brainerd, Minnesota. I was at that meeting. The first session started with a series of generals and scientists who got up and delivered 15- to 30-minute talks on various aspects of the Air Force of the future. They discussed the new fuels, new radars, new electronic equipment, engines, bombers, missiles, atomic weapons, etc., but never a word about concrete and reinforced steel or any substitute thereof. I also gave a talk which I started by saying, "There has recently been released a picture called 'SAC.' The ostensible stars are June Allyson and James Stewart, but the real lead of the picture was without question the B-47 bomber. Now I am hoping that 10 years from now we will find a new picture being released called 'SUC'—Strategic Underground Command."

The first scene of this picture might start out as follows:

Three fellows are sitting around a table playing pinochle in what is obviously an underground shelter. There is suddenly a terrific shaking.

One of the fellows gets up to look at a meter.

The other asks, "How much was it?"

The first says, "About 100 megatons."

The second says, "I can always tell by the shaking."

Finally the third one says with a great deal of irritation, "Write it down on the morning report and sit down and deal."

During this entire scene, there would be a teletype operator in the background, calmly punching away at his machine and receiving messages in reply.

Now this is admittedly asking for a lot, but I don't think it is inconceivable. In any case this is what we are aiming at.

PEACETIME CRITERIA VERSUS WARTIME PERFORMANCE

J. J. O'Sullivan
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Changes now rapidly taking place in modern weapon technology and warhead design demand a complete re-evaluation of the design procedures employed by architect-engineering organizations when considering protective construction. Peacetime procedures and criteria will be found inadequate for solving the problems we will face in the design of underground installations.

It has been said before, but it needs repeating, that a revolution in military technology occurs every five years. Weapons change in two to three years, fighters and small missiles in three to six, and bombers and large missiles in four to eight years. If the designing civil engineer does not understand the significance of these changes, he will always be a few revolutions behind and his designs at best may be inadequate and expensive. In most cases the plans will be obsolete before they are off the drafting board. The engineer, therefore, must re-examine his peacetime design criteria, scrapping that which is no longer useful for the peculiar conditions he will face, and develop new criteria to fit this new design era. This will not come easy because by training and experience we engineers are conservative and accept changes slowly. For example, consider the prospect of changing a building code. If you are an engineer developing a new product which you wish to be accepted by the building code, you are impatient with any delay. If, however, you are the municipal engineer who must be responsible for the safety of the public in buildings, you will want to be completely convinced (and this may take years) that any change is for the better and that no safety problems are being slighted.

Any engineer who wishes to change our peacetime procedures must overcome similar inertia. Yet it is vital for our country that we push through these changes. You must be the engineer with the new product, demanding a building code change.

I suggest that we re-examine some of our time-honored standards and approach our design tasks with a flexible mind. We must develop in the engineers a desire to think up new solutions, unusual solutions, even fantastic solutions. As an example of this, I might refer to Dr. Brode's water door which will be discussed later in this Symposium. Dr. Brode uses ordinary water as a building material under conditions where the average engineer would ordinarily consider the use of steel or reinforced concrete. His use of water as a blast door to protect the interior underground areas from blast demonstrates that it is possible to use a completely different material to solve a standard engineering problem. An engineer must bring a free imagination to the drafting board when designing underground installations. Otherwise his solutions will be forced adaptations of aboveground peacetime structures developed over past years for entirely different climatic conditions and use requirements. After the compromising changes are made for deep underground conditions, they certainly will be inefficient and most likely inadequate and expensive.

As another example of approaching a standard problem in a new way, we might consider the blast-sheltering of airplanes. Some years ago many of us in various parts of the country were investigating the possibility of protecting aircraft from high overpressures. Most of us approached the problem by "beefing up" standard aircraft hangars for use either aboveground or below with a ramp to provide hangar access. Thus, we were merely continuing the standard evolution of the aircraft hangar from the farmer's barn by making one more slow step forward. We ignored completely the most important element--the feasibility of designing a beam to carry high blast loadings of a length greater than the wingspan of the airplane. At this point William Burk proposed a novel solution. He designed a shelter in which the roof slabs bridged the width of the wing instead of from wing tip to wing tip. This was accomplished by wrapping the walls of an underground concrete shelter around the planform of the aircraft and lowering the plane into the shelter through the roof by means of an elevator. The roof was formed of horizontal sliding concrete doors. Thus, with relatively simple design the aircraft shelter was made feasible.

The present practice when designing a "hard" structure is to strengthen a soft building to resist the high overpressures, then place it below ground. I maintain that such an approach is wrong. It is better to design a completely new building by rewriting the criteria, starting with the function of the structure, especially in its blast environment. In some cases such studies will show that it is possible to design a hard structure to resist a reasonable level of overpressure at a cost equal to that of an existing soft structure. While these two structures will perform the same functions, I do not claim that the floor area of each will be identical. This leads to a very interesting principle in the economics of underground construction. Construction cost depends roughly on size and volume. If you cut volume you automatically cut cost. Therefore, in addition to the engineering duties of a designer, he also must be aware of the effect of construction costs on the acceptance or rejection of a hardened structure. In some cases, if costs are ignored during design, the hardened structure may be eliminated in favor of an unprotected soft structure. The engineering designer must understand that cost is an important part of his solution to the base problem. If he can build cheaply, then it is possible to buy more bases, missiles, or aircraft for the same amount of money.

Since costs are important, and since they depend on space, the civil engineer and architect lacking experience and training in space-saving might employ as consultant a naval architect when designing underground installations. Ship designers make use of many space-saving tricks, such as the dual use of space; for example, an officer's living quarters serve as his daytime working office. The mess hall during the day is the movie theater at night. These designers also design in three dimensions: some items are hung from the walls of the structure, while others are placed on the floor in the conventional manner--still other items are mounted above those on the floor in "piggy-back" fashion to make use of the empty volume found above most machinery in standard layouts. Ceiling heights are designed to provide adequate sound insulation and ventilation, but not according to shore-type conventions. I do not recommend cramped quarters, for obviously the peacetime operating efficiency will decline, nor do I suggest that the above examples could be successfully used in deep underground installations, but I do believe that serious investigation of methods to eliminate waste space should be made.

The civil engineer should approach his design problems with the same ingenuity and freedom of thought that the aircraft designer brings to the design of a bomber and the missile engineer to the missile, understanding that he is responsible for a very important part of the military posture. If a missile installation does not perform adequately, no matter how expensive or sophisticated the design of the missile, it will not arrive on target.

In standard peacetime construction the engineer plans his structures to work within the elastic limits of the materials and standard building codes. For architectural reasons he seeks to eliminate elements that sag and crack under loads. The engineer designing blast-resistant structures is not or should not be interested in such peacetime criteria for design. The ability of his structure to protect the vital functions of its contents, enabling them to perform their wartime missions, is the one design criterion which must be established. The wartime functioning of the shelter is the one important element in design. Such wartime performance standards will allow buildings to move, walls to tilt, roof slabs to bend and crack, provided the installation's fighting ability is not impaired. Moreover, in some cases we may wish some parts of our structures to collapse under the first enemy weapon. Thus we might design the tunnel entrances to fail on the first blast loading so that the installation may resist a significant number of multiple blasts.

In shockmounting equipment for use in ships, tanks, and other military vehicles, it is customary to design the shockmounting for individual units, test it in the laboratory, then install it in the vehicle and put it through a series of field tests. When failure is observed at any point, this element is redesigned to pass the field test. These tests give us a high degree of assurance that the equipment will operate as designed. The shockmounting of military equipment in shelters poses a different problem; here we must design the shockmounting for conditions which contain a number of unknowns. To provide for such unknowns, we must add numerous factors of safety. Laboratory tests provide some confidence in our design, but we have no means of truly testing the reliability of this equipment.

When faced with such uncertainties, we may wish to solve the design problem by running away from it rather than meeting it head-on. Thus we might consider the alternative of using the money budgeted for shock-mounting for buying greater depth for the underground installations. This again is a survival versus cost problem.

If we look at other fields of engineering, such as electronics and missile engineering, we will observe that the significant technical break-throughs since World War II have been made by small contractors pushing ahead on small parts of the over-all problem. In the field of civil engineering, progress is much slower. Significant breakthroughs are quite rare, and the average engineer has no feel for the areas in which work should be done to be of use in the military program. For this reason most of the work in the field of protective construction must be led, financed, and programmed by the government agencies. This is probably not the ideal method because of the lack of flexibility required for proper research in the field of engineering.

If we examine current procedures by which a hardened structure or an underground installation could be designed, we will find that the steps would be somewhat as follows: One or more men in a government organization are called upon to prepare design criteria to be used by an architect-engineer firm for designing a base. The designers prepare the plans for the blast-protected shelter or underground installation, making use of this criteria. They submit the plans to the issuing agency, which in turn lets construction contracts.

For peacetime construction this system is adequate. In general, we know what we want; we have designed and built similar structures in the past; we have a good idea of what the finished product should look like and how it should operate. The architect-engineer's solution is quite suitable. This procedure, however, does not suit our needs for the design of underground installations. With few exceptions, the design of underground installations is a new field in which clear principles and criteria are not available. The man who prepares criteria may be uninformed as to the interaction of his criteria with engineering design decisions. Some details--for example, the width of a room--may be nothing but a copy of an aboveground structure. Engineers in this audience will suspect that not one width of room is the proper solution, but perhaps some dozens of different sizes would serve the operational functions adequately. The architect-engineer may feel he must restrict himself to the dimension spelled out in the statement of criteria. Yet, room sizes, areas, and heights should not be based upon the first solution that occurs to a planner. The architect-engineer must search behind the given criteria and offer suggestions for changing these criteria to provide better solutions. He must consider the interaction of weapon effects, planning, design, and construction. This requires him to develop optimum shapes for underground installations with the aid of leading tunnel contractors and mining engineers, so that the contractor can use his men most efficiently. This may mean some compromise, with the planner altering underground installations to fit common construction techniques.

The type of underground structure which we place in the underground chamber has an effect on the over-all construction time and cost. The designer must weigh a number of solutions to determine a reasonably good design which recognizes the interaction of excavation and choice of underground building. The number of floors in an underground building is a function of the excavation cost and the building cost. Geology, too, influences the shape and layout of the underground areas; for example, in a limestone area we might cut the ceilings of the underground chambers flat, whereas in hard rock we might use arch shapes. Different excavation techniques would be used, and in turn, the building would be modified to fit the excavated cross-section. Geology would also influence the choice of the shapes and sizes of the excavated chambers.

The engineer, while making preliminary studies, should also be examining the criteria provided by the manufacturers of the equipment to be installed in the underground buildings. He should convince himself that the manufacturer's requirements are necessary for proper operation of the underground equipment. His purpose should be to determine the amount of flexibility available to him in design. The design of utilities, including the type of power-generating plant, the location of the oxygen inlet shafts for the prime movers and their relation to the air shafts required by the contractor for efficient drilling, blasting, and excavating of the underground chambers should be studied carefully to determine their influence on the over-all design. Many other facets of the general problem must be considered. From the few examples I have given you can see that the engineer in the preliminary planning will develop dozens of adequate solutions from which he must choose one. This may be the minimum solution, but not necessarily so.

The engineer should provide the monitoring agency with a minimum solution, making clear all the facts which he has collected and his reasoning on choices. The military planner should understand the minimum solution, its cost and characteristics, as well as all the additional refinements and betterments, together with their construction costs and time, so that he may choose a proper combination of minimum solution and betterments. This solution will not depend upon cost alone, but on many facts known to the planner but not available to the engineer.

The most important decision which an engineer must make in planning underground installations for warfare is in determining the functional plan. This includes sizes and length of tunnels, configuration and cross-section of the underground area, depth of cover, etc. Since original planning is so important, it would seem reasonable to devote our best efforts to this phase of the problem. This remark will appear to be so obvious that you will wonder why it was made. Yet the speaker knows of cases in other fields of protective construction where detailed design of the hardened structure was started five days after awarding the engineering contract. Even in conventional construction, the planning phase is sometimes not given proper emphasis. This was pointed out recently by a report to the city of Toronto by A. D. Margison, professional engineers:

> In any capital undertaking, private or public, the major factors affecting value to be received for the expenditure to be made are rooted in the decisions made pursuant to the adoption of a functional plan. Frequently, inadequate attention is given at that stage while much subsequent attention is given to possible savings by economizing in construction methods, materials and workmanship. Attention should be given to expenditure at all stages of an undertaking but the major savings ensue from the initial establishment of that functional plan providing the desired results, having proper regard for all factors, at the lowest cost.

In developing underground installations, it is important to define the problem. Part of the problem is the threat. Too often in the past the design of military structures was based upon the enemy threat at the time the plan was proposed, rather than that which would exist when the structure was completed. In some cases these structures became obsolete before construction was completed; therefore, we must clearly define our design problem. What is the purpose of our structure--what enemy threat are we trying to resist--through what years do we wish this installation to survive? In formulating the general problem, we must consider the importance of the structure. If we plan to protect a central communications unit on which many weapons and fighting installations depend, whose destruction or elimination would cause great damage to the military posture, then this becomes a prime enemy target. The designing engineer should be aware of this. He should understand the effort an enemy might make to destroy this individual underground installation. This will influence many of the purely engineering decisions he makes when designing the structure.

The designer must determine what are the vital functions of the installation which must be protected. He must separate from the over-all plan all those peacetime items not necessary for wartime operation. A vital element is one which is necessary for carrying out the wartime function of the base. Obviously some compromises must be made at this point. However, the engineer should have a clear understanding of what he is compromising. Some elements which are required only for peacetime operation might be placed in the underground areas because they add to the efficiency of the day-by-day operation of the installation, even though they would not be used during wartime. Under efficiency we must consider the elements of habitability and cheerfulness of surroundings in which men will work during the peacetime period.

The engineer must be briefed by the military officer on many of the problems which are purely military and operational, and he must be made to understand that the success of a wartime mission in many cases depends upon his intelligent solution of architectural and engineering problems. Because his interest is in engineering design, an engineer will often rush into the design phase, following standard design techniques. Too often engineers have no interest in military and operational aspects of the problems.

An engineer must design into his structure a capability for increasing its strength. For a deep underground installation, this capability must be limited to the tunnels, shafts, doors, and closures. You cannot increase the strength of the chambers in the underground installation. Therefore, it is well to locate the underground installation at a depth below ground sufficient to give the protection required to this area for the full life expectancy of the installation. This is an irrevocable decision. Once you dig the chambers, you cannot increase their strength. Your only choice is to abandon the structure and build a new structure. Doors which would be set some distance back from the entrance of the tunnel could be strengthened at a later date to meet an increasing enemy threat. Outside communications and the airshaft covers and the like also could be strengthened at a later date.

This audience is familiar with the present procedure by which the Air Force awards architect-engineering contracts. Because of the interaction of many elements, such as military planning, weapons effects, reaction of rock chambers to high blast loads, and others which are peculiar to a deep underground installation, it might be advisable to consider other methods for handling design contracts for deep underground installations. One method might be to consider the use of part-time advisory committees. Let me explain three ways in which such committees might be used.

First, assume an architect-engineering firm is hired to design structures based upon the criteria provided by the client. The engineering firm, upon completion of its work, presents its plans to the advisory group for review. This sometimes results in a design which is pedestrian at the best, and at the worst could be wrong, requiring extensive re-work and changes with significant loss of time.

Another way is for the engineering company to use the advisory committee as consultants. In this case the engineering company designs those things which it has done before and calls upon the consultants for specific answers to specific problems for immediate design use. This is probably the worst way to use an advisory group. The architect-engineering company has no pressure on it to do a good job, or even to hire responsible people, because it assumes that it can get all its answers from the consultant. The consultant is in the peculiar position in which he is only working on this problem part time. He is busy on other projects and other problems; therefore, in general, no one in either group is working on the over-all problem at any time.

The third way, and in my opinion, the best way, is to use an advisory consulting group almost on a full-time basis--to assign to it real responsibility for parts of the design, so that they may be able to carry through from start to completion that portion of the design in which they are truly expert. This means that good thought is being put on the proper problem, and responsibility is being placed on the consultant so that we can be sure that certain difficult portions of the general problem are being looked at by people with the proper background and by the proper continuity of thought, and we may expect that good work will result. It is possible that we may want to add another factor of safety here--by using a second group of consultants to check the first group.

It must be made clear that we need many good minds employed in new work of the type we are discussing. In this way we can solve the problems that are facing us in the design of underground installations.

NUCLEAR BURST PHENOMENA PERTINENT TO DEEP UNDERGROUND STRUCTURES

H. L. Brode
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On the face of it, this title appears to be self-contradictory--since if a structure is deep enough underground no nuclear explosion effects will be pertinent to consider. The most likely phenomenon to have any influence deep underground would be the seismic signals--the nuclear radiation, the air blast, the thermal radiation all would be of no consequence.

But any realistic underground complex for human occupancy on either a military or civil mission must have certain connections with the outside world. Provision will be necessary for normal ingress and egress, for ventilation in normal circumstances (not during attack), for communications of various sorts, for utilities (power, water, sewage) and for eventual (post attack?) evacuation. So that none of these presents an Achilles heel we will briefly consider here those phenomena which may have significance from the standpoint of surface structures and equipment as well as for the deep underground structures.

We shall for illustration consider a one megaton surface explosion. When such a nuclear weapon is detonated, the nuclear reactions responsible for the impressive sudden release of energy create equally vast amounts of ionizing radiation. Only the gamma-rays and neutrons can escape the bomb and go to any appreciable distances in air. The other radiations and even a large fraction of the neutrons and gamma-rays are absorbed in the bomb itself or in the air immediately around the bomb. Where fission reactions are involved in the bomb, radioactive fission fragment nuclei are created which then contribute to the activity and doses over a long period of time--while the neutrons are generated immediately and disappear within a fraction of a second.

The amount or intensity of radioactive effects is generally measured in roentgens or roentgen equivalents. A roentgen is a measure of the energy per unit mass deposited in a material by the ionizing radiations.

A dose of 25 r or less does not cause serious biologic consequences in men. A dose of 450 r will kill 50% of those exposed within a month after exposure, and a dose of 700 r can be expected to cause 100% fatalities. Figure 4 illustrates the gamma-ray doses to be expected within two miles of a one MT surface burst. Also indicated are the approximate peak overpressures at these distances.

Various materials can be used to shield against or to reduce these dosages. In Fig. 5 the thicknesses of some typical materials are illustrated to show how much is necessary in order to cut the dose in half. Taking twice these amounts will reduce the dose to a quarter of its initial value, etc. Thus the 200,000 r at one/half mile (200 psi) can be reduced to less than one r behind nine ft of concrete or perhaps more realistically reduced to 50 r by six ft of concrete, or by eight ft of earth.

Virtually all the neutrons are created in the fission or fusion reactions within a microsecond and diffuse away in a few hundredths of a second. Some neutrons captured in the nitrogen in the air lead to further gamma-rays--(these gamma-rays are included in gamma-doses). Damage by neutrons cannot be measured quite directly in terms of ionizing effects--neutrons are neutral (uncharged) particles as the name implies, and so they cause ionization only indirectly by interaction with hydrogen nuclei and others. A "biological equivalent" must be included which indicates the neutrons to be somewhat more damaging (~1.7 times more) than their ionizing capability would indicate giving a dose in "rem" (roentgen-equivalent-man).

Neutron doses depend on weapon design (unlike gamma-rays which depend mainly on fission yield)--so only a typical case is considered here. The expected doses within two miles of a one MT burst are shown in Fig. 6 along with the gamma-ray doses and the approximate overpressures. Note the apparent crossover where neutrons are more effective relative to gamma-rays at very close-in distances. This is a real effect since there are many more neutrons than gamma-rays generated at the bomb, but the neutrons have a shorter mean free path in air--i.e. get scattered and absorbed faster than the gamma-rays--so become relatively less important at increasing distances.

In order to shield against these neutron doses we need material which will slow down the fast neutrons so that they can be more easily captured by nuclei. Light elements or elements of low atomic weight are most effective at slowing down the neutrons. The slowing down is a matter of colliding with atomic nuclei and giving up some energy at each collision. If a billiard ball were to run into another billiard ball it can lose a large fraction of its velocity or kinetic energy, but if it runs into a bowling ball it bounces off with almost the same velocity and gives the bowling ball only a slight nudge. Neutrons are slowed down most effectively in hydrogen or hydrogenous materials such as water or plastics or wet earth or even concrete because the neutron and hydrogen have similar masses and so more nearly equally share the energy of each collision.

When a neutron has been slowed down it is much more easily captured in a nucleus. A good shield must provide light nuclei for slowing down neutrons and appropriate nuclei for capturing them. And since such captures often lead to gamma-ray emissions, we will also need a certain amount of gamma-ray shield. For shielding against gamma rays high atomic weight elements are most effective, so a compromise in the nature of the shield is usually necessary.

Earth or concrete are fair compromise materials. About 10 inches of concrete reduces these neutron doses to 10% of their initial value--20 inches to 1%, etc. By adding iron punchings in modest amounts to the concrete mix--(iron being a fairly high atomic number element good for gamma-ray absorption)--the shielding quality of concrete can be improved so that seven inches of concrete will accomplish a 90% attenuation, i.e., 10% of dose transmitted. Further, by including boron compounds such as colemanite which have very large neutron capture cross section an even more efficient shield can be constructed.

These impressive surface doses can cause trouble to electronic equipment--particularly transistorized circuitry--unless they are adequately shielded. However, an 18-inch shield can be adequate at the 100 psi point for most such equipment.

It is abundantly clear that a deep underground structure is not subjected to any measurable dose--and we need be concerned with initial or delayed radioactivity only in so far as it applies to surface connections to the underground complex.

Fireball--The release of one megaton of energy (10^{15} calories) in a time much less than a millionth of a second and in a fairly small mass and volume leads to temperatures of millions of degrees, and much of the energy is in the form of radiation which diffuses out of the bomb and into the air.

The radiation from such high temperatures is in the form of X-rays and ultraviolet light mostly--and these do not go to large distances in air. The air immediately around the bomb is heated by absorbing this radiation to temperatures also in the neighborhood of a million degrees. Air at temperatures above 300,000° C is quite transparent even to X-rays and ultraviolet--so that the radiation fills up a sphere of air uniformly, with nearly equal temperatures everywhere inside and with cold air outside.

This isothermal sphere grows by this radiation diffusion process faster than hydrodynamic shocks can move until it has cooled by expansion into a mass of air such that its temperature is about 300,000° C. Then a shock forms and further expands the fireball by hydrodynamic motion. This shock is a very strong shock--and it heats to incandescence the air that it engulfs, although the shock temperatures are well below the initial interior temperatures.

At the time the shock has decreased in strength to 1000 psi (.077 sec after burst), the blast radius is about 1500 feet for one megaton. In this short time the interior temperatures have dropped by a factor of ten to around 100,000° C, and the apparent fireball temperature--i.e. the temperature of the shock front--is only 3000° C, and is beginning to become transparent. The surface of the ground then is covered with a hemisphere of high pressure and high temperature over an area 3000 feet across. The shock is so strong as to make the air luminous. Beyond here it begins to leave the luminous fireball behind--and as both shock and fireball further expand, the intensity of thermal radiation increases because the hot fireball becomes more transparent to visible light and is thus drained of its considerable store of heat energy.

This thermal radiation presents a hazard only to the surfaces of exposed structures or machines. At close-in distances as at this 1000 psi point, the air passing over it is super heated--rising to as high as 40,000° C and lasting for seconds--but at distances beyond the 100 psi point the air is not directly heated appreciably--and only the heat shining out of the fireball makes the surfaces hot. The 50 psi point for one MT receives about 1500 cal/cm^2, and the 20 psi point gets about 400 cal/cm^2. About 80% of the total thermal pulse is over in less than 10 seconds. As a result, although a relatively large amount of heat is deposited on the surface of exposed objects, it does not have time to penetrate to much depth and will be of little consequence to anything but the first few inches of concrete. Exposed steel may be melted on the surface but in thicknesses necessary to withstand the blast the heat will be dissipated without contributing significantly to structural failure.

Thermal effects are of secondary importance to the deep underground structure. Only the design of openings, vents, and surface support facilities may be somewhat affected.

Cratering Rupture and Ground Shock-- Of considerably more interest to the deep underground installation are the phenomena of cratering, rupture and ground shock.

The origin of ground shock waves which will reach to extreme depths of burial lies at the surface and presents a fairly complex set of initial conditions. The air blast wave, which I have briefly described, supplies a respectable load extremely quickly over a large area of the ground surface. These pressures are transmitted down and out in the ground at roughly seismic velocities (several thousand feet per second) with decreasing amplitudes as the dissipative features of the soil and rock extract energy from the wave.

At the same time some considerable energy enters the ground directly from the expanding bomb vapors which carry impressive momenta and create extreme pressures in a limited region below the bomb.

The effect of the air blast is roughly that of a compaction force on the surface--like striking the surface with a huge circular disk--an expanding disk--and one with higher pressures at its periphery. The energy entering the ground from the impact of the exploding bomb mass is localized and acts much like the impact of a high speed pellet, blowing a crater out of the soil by hydrodynamic action.

Survival of an underground structure is not contemplated within the crater--but is quite likely only a short distance outside the crater.

Beyond the actual crater will be a region of crushing, of plastic flow and permanent displacement--extending as far as two and a half times the crater radius. Structures can survive in this region in spite of high acceleration forces and permanent displacements in the soil.

Relatively small, resistant structures can roll with the ground shock. The shock, being of long wave length and having lost some of the sharpness of its front, acts as an ocean swell would act on a cork float--shaking it and moving it but neither tearing nor crushing it with any high local stresses. However, a deep tunnel or cavity may be damaged by the spall resulting from high stress developed if the wave retains any high frequency or sharp components. In general, the high frequency accelerations are damped more rapidly than the lower frequencies. After a ground shock has traversed a considerable mass of earth it has lost its sharp characteristics, and has greatest amplitudes and soil velocities in the region of one, two or three cycles per second beyond two or more crater radii.

Relatively brittle or inflexible large or long structures like pipelines may be damaged seriously out to as much as three crater radii.

For one megaton surface burst the crater dimensions will depend on the nature of the soil and to some extent on the bomb. A representative depth might be 145 ft and a radius of 650 ft. The rupture zone may extend about a half a crater radius further in most soils. For a surface burst some of the residual radioactivity will remain in the throwout in the vicinity of the crater and lip and will constitute a serious health hazard to human activity above ground for a considerable period of time. About half of the total radioactivity falls out within a few hundred miles--mostly downwind. The larger particles carried up in the wake of the rising cloud formed by the hot remains of the fireball will fall back around the crater. These larger fragments and dust particles will contain much of the activity.

Typical dose rates in the immediate area (within 10 or so miles) will run from a few thousand r/hr in the first few hours after burst to a few hundreds at the end of a day. Total doses (integrated over time) after 18 hours may be in excess of 3000 r over a thousand square miles.

Clearly, surviving nearby surface installations or support structures will not be habitable for many hours after a megaton weapon surface burst even in extreme emergencies.

For many minutes after a surface burst, heavy clouds of dust and dirt will be in violent turbulence over the surface. The strong updrafts in the rising stem below the cloud will carry aloft much crater material and drop it as it reaches the base of the cloud or exhausts its upward momentum. Such material may be of appreciable size and may constitute a rain of rocks and pebbles for many minutes.

It seems fair to conclude that one should not be in a hurry to leave a deep underground structure after a surface or air nuclear explosion.

DISCUSSION

MR. DANA A. BENSON (Rome Air Development Center, Griffiss Air Force Base, New York): Have you negated the possibility of underground bursts and their effects?

DR. BRODE: I didn't intend to negate this. I restricted attention here to a surface burst. We could also consider air bursts and the deep underground bursts. If, however, we are going to large-yield weapons, depth of burial has a decreasing importance as the yield goes up because it scales to something closer to a surface burst. True, there may be important differences in the ground waves, if a burst is deep. In general, however, penetrating projectiles do not penetrate more than something like fifty feet. This is not a very large depth on the megaton scale of distances, however, and the phenomena would be essentially the same as I have described for the surface burst. I certainly would not want to rule out the underground burst.

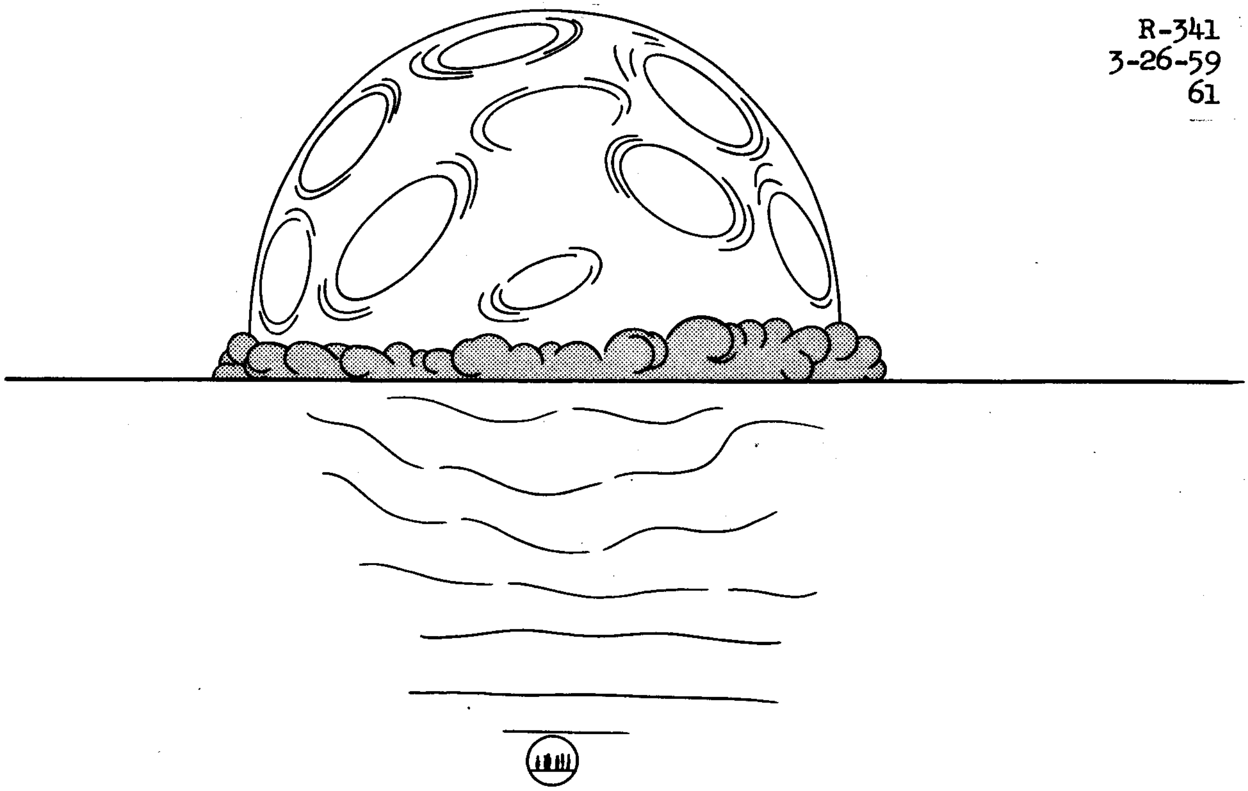

Fig. 1 — Surface burst vs underground structure

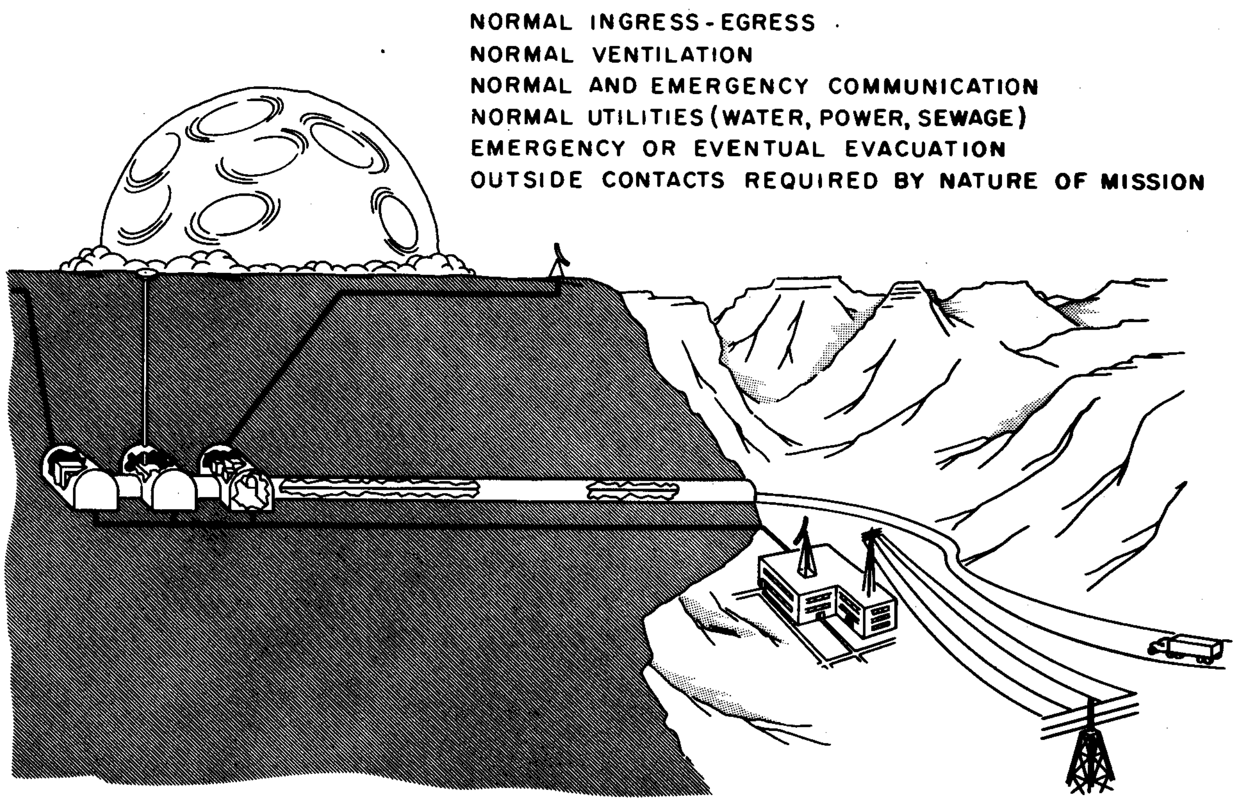

Fig. 2 — Underground installation

FIG. 3—ONE MEGATION-SURFACE BURST

SEQUENCE OF PHENOMENA

NUCLEAR RADIATION

ISOTHERMAL SPHERE

AIR SHOCK

THERMAL RADIATION

CRATERING

GROUND SHOCK

CLOUD RISE AND THROWOUT

FALLOUT AND AFTERWINDS

FIG. 4—GAMMA RAY DOSE FOR ONE MEGATION

~40 R	AT	2 MILES	(~10 PSI)
~500 R	AT	1 1/2 MILES	(~20 PSI)
~10,000 R	AT	1 MILE	(~40 PSI)
~200,000 R	AT	1/2 MILE	(~200 PSI)

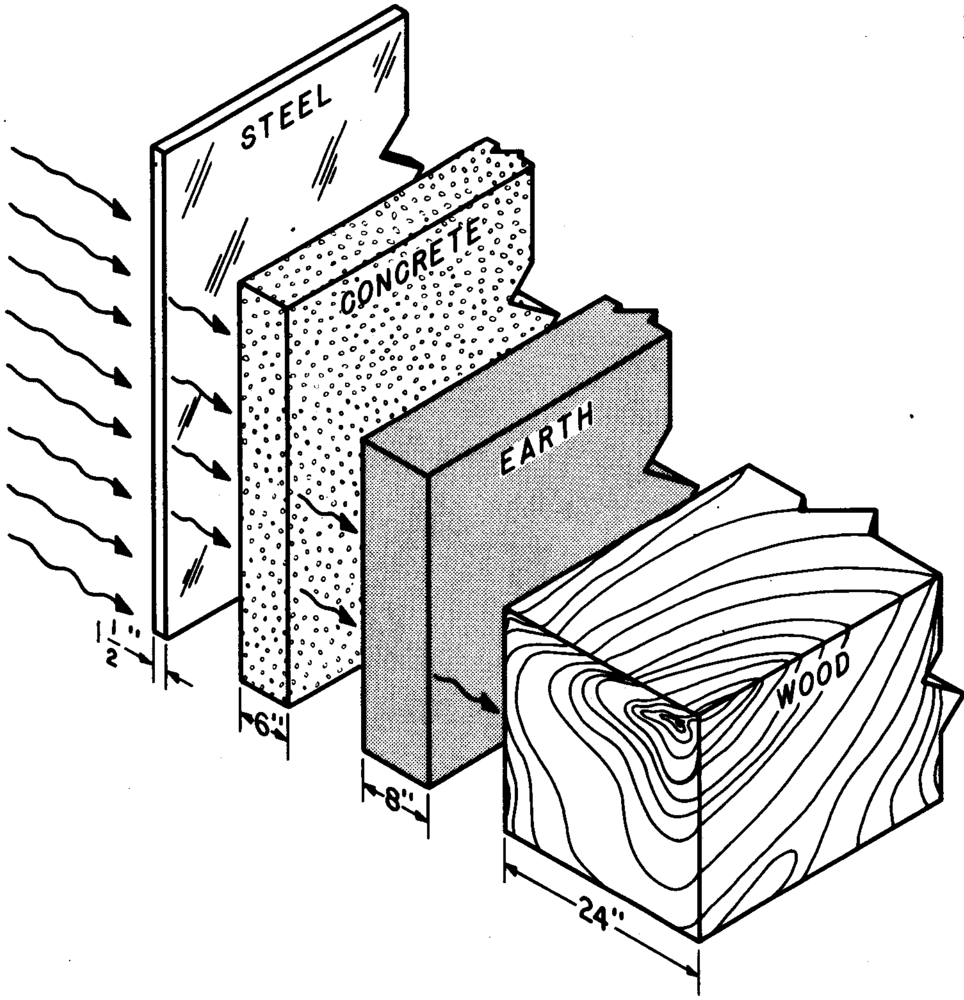

Fig. 5—Half-thicknesses for fission fragment gamma rays

(γ-dose)	Neutron dose		
(~40 r)	~1/2 rem	at 2 miles	(~10 psi)
(~500 r)	~ 20 rem	at 1 1/2 miles	(~20 psi)
(10,000 r)	~1,800 rem	at 1 mile	(~40 psi)
(200,000 r)	~ 330,000 rem	at 1/2 mile	(~200 psi)

Fig. 6—Neutron dose for one megaton

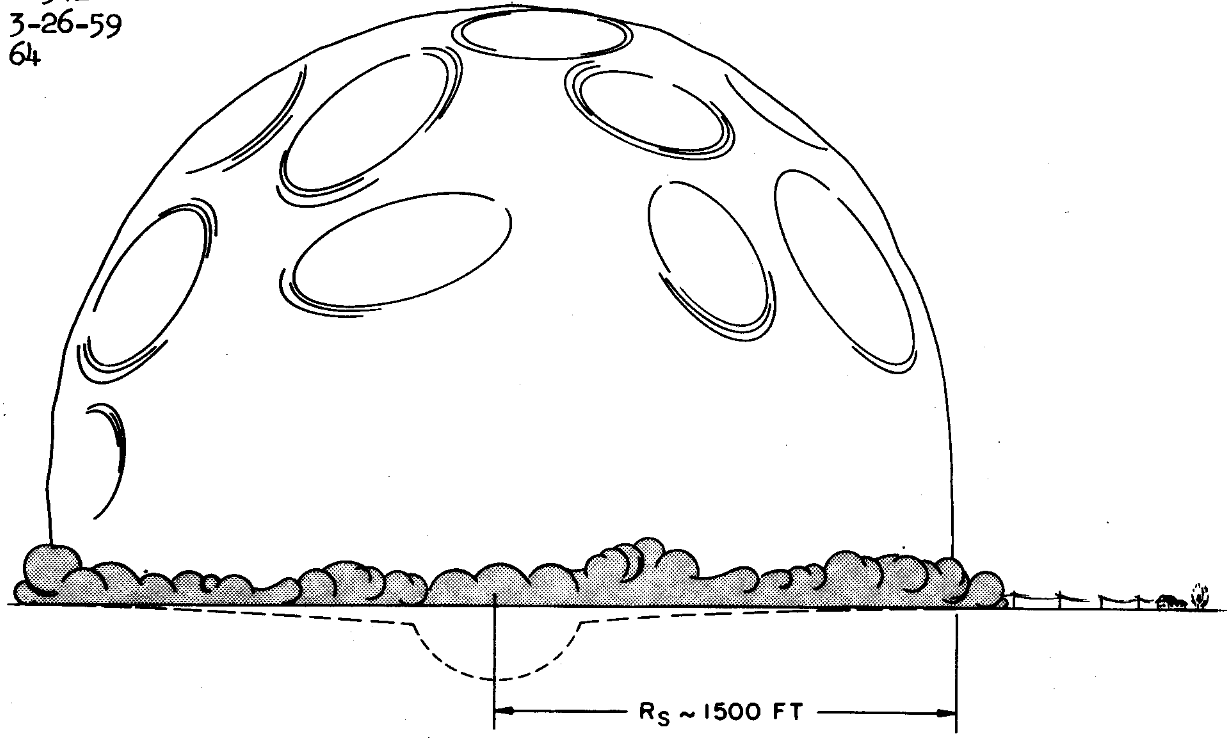

Fig. 7 — One megation - 1000 PSI
(.077 seconds; 1500 ft radius)

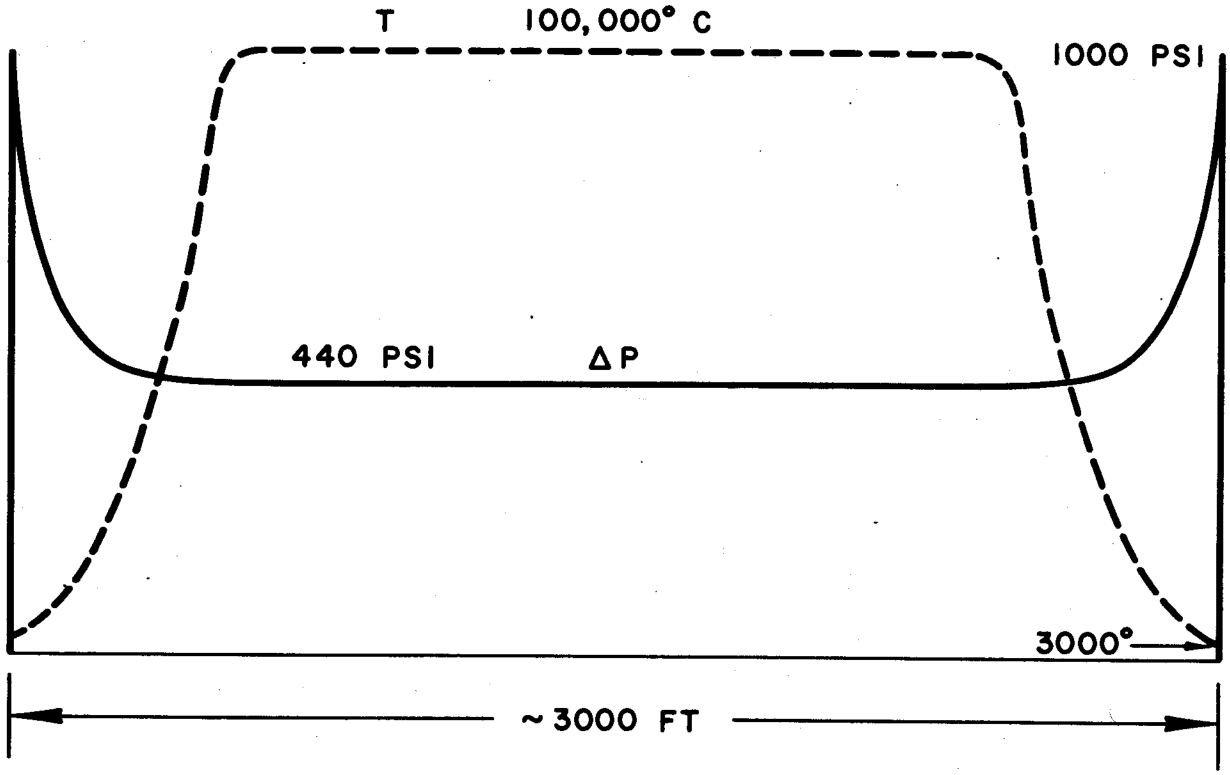

Fig. 8 — One megation - surface burst

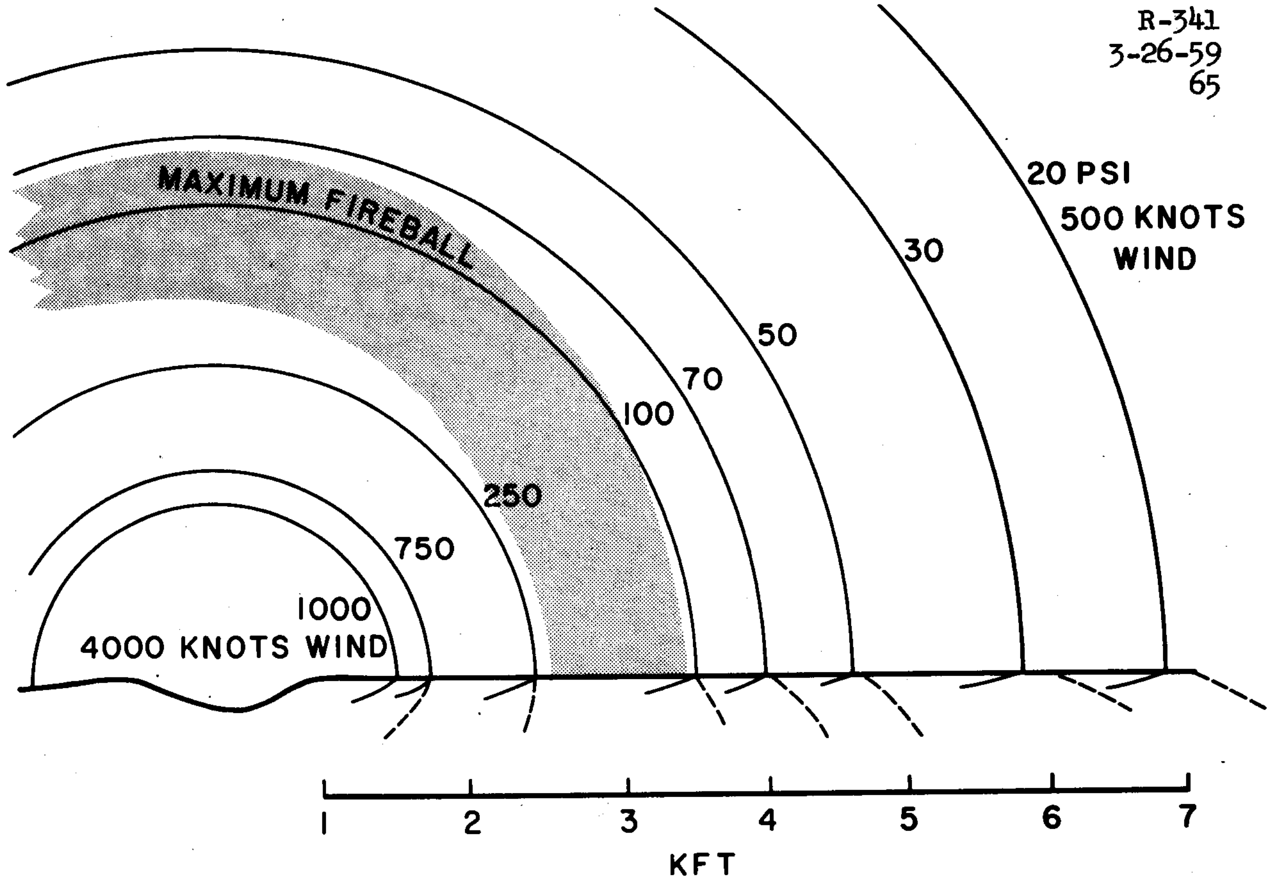

Fig. 9—Shock pressures as function of distance from point of 1-MT burst

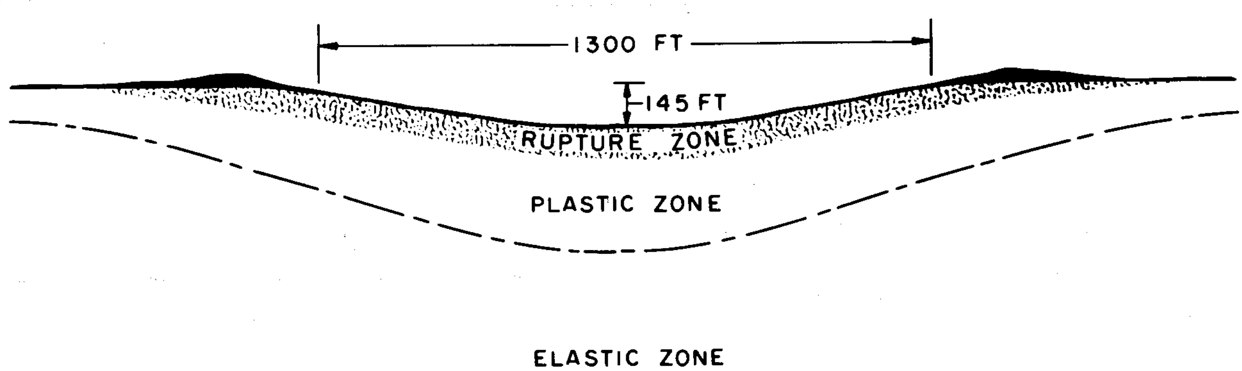

Fig. 10—Schematic cross-section of typical crater, 1-MT burst

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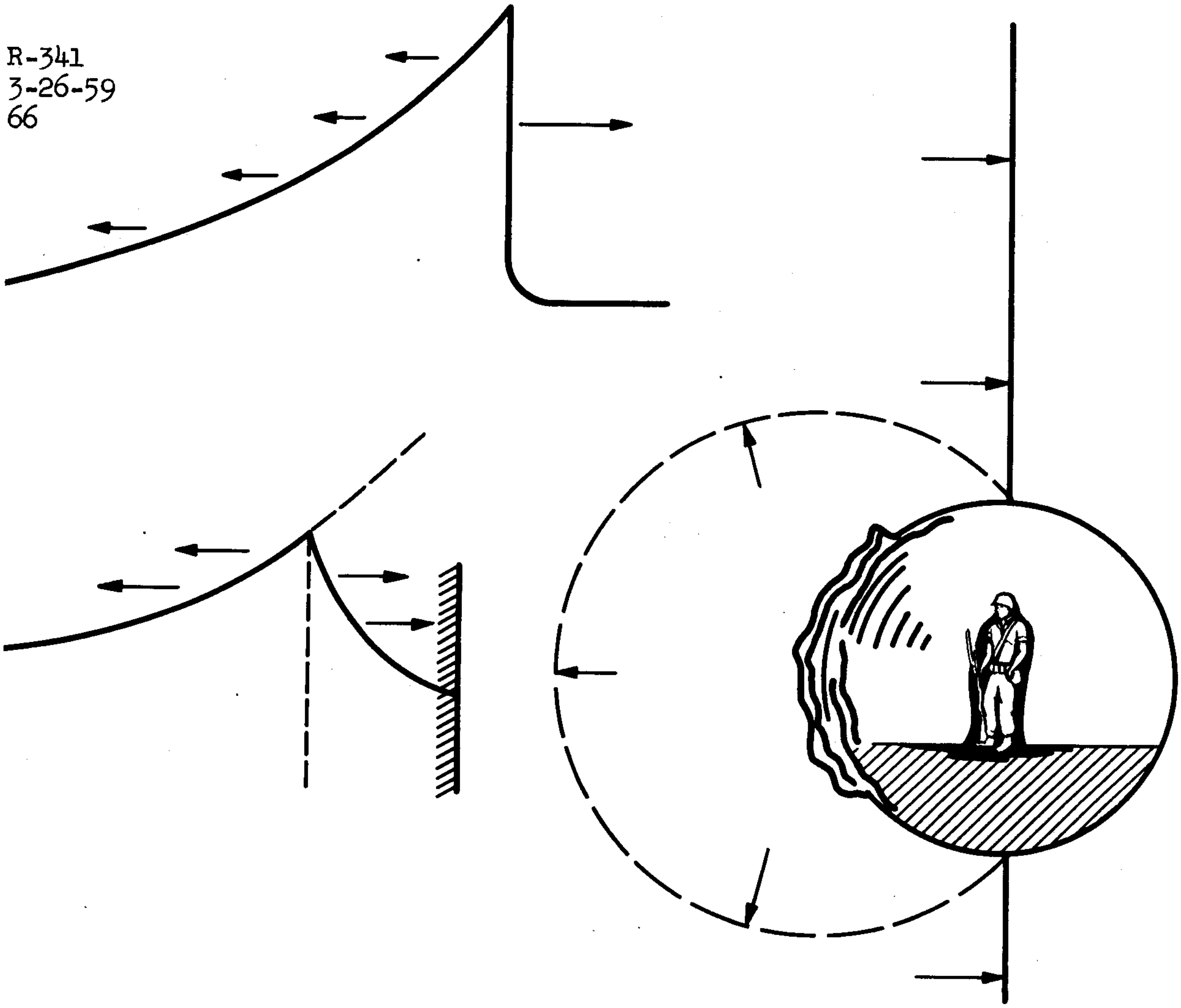

Fig. 11 — Spallation

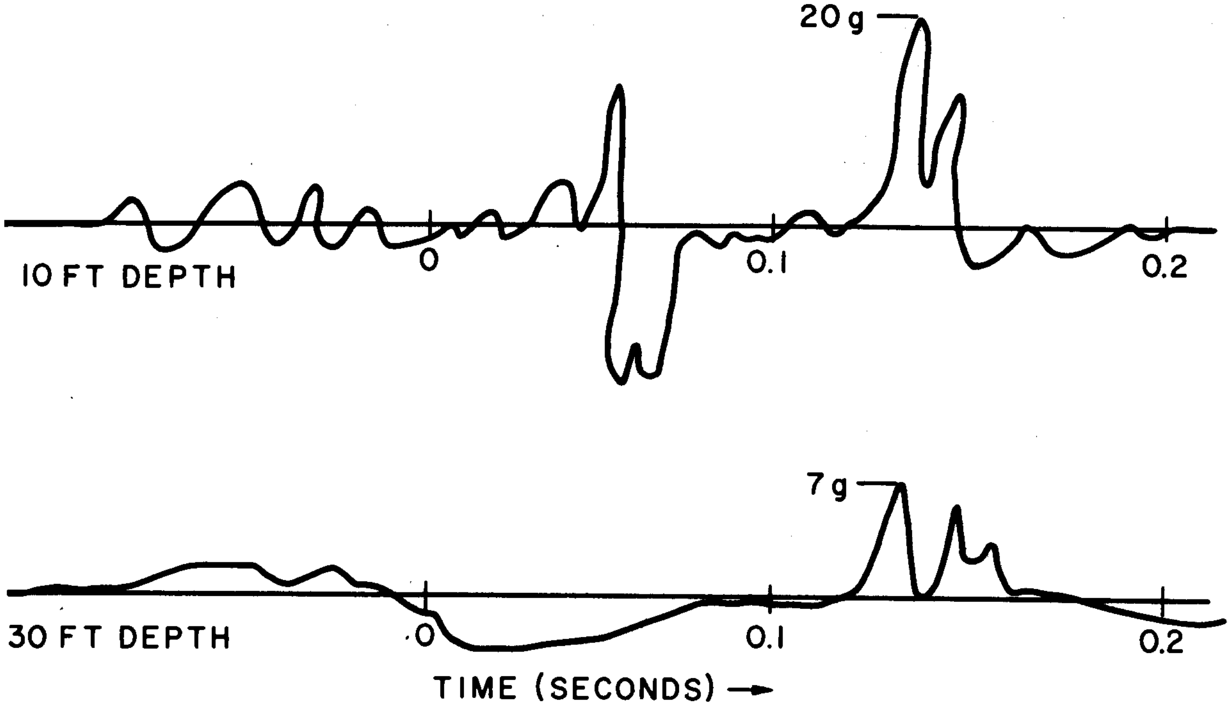

Fig. 12 — Vertical accelerations at shallow depths

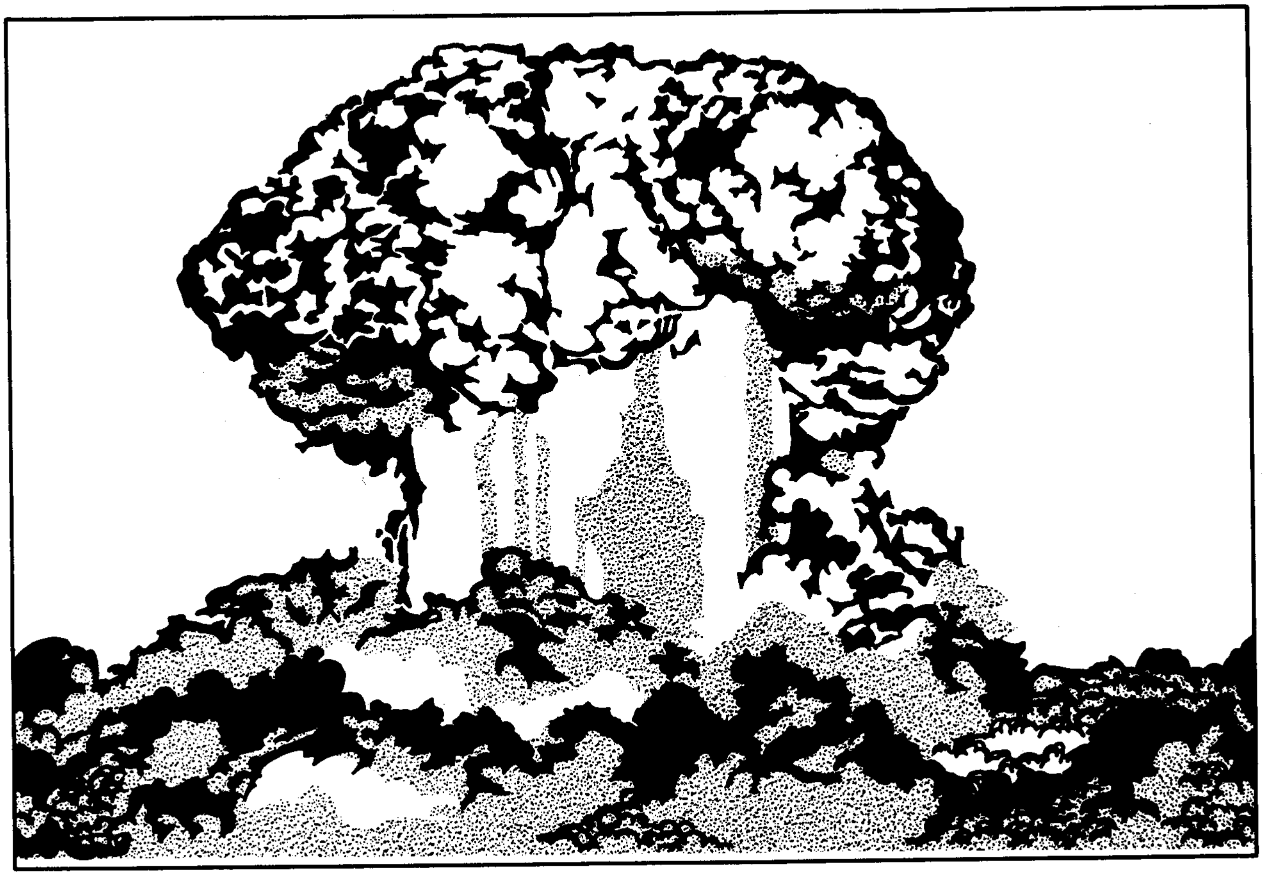

Fig. 13

COMMUNICATIONS VULNERABILITY

J. B. Carne
The RAND Corporation

In the discussions thus far we have considered the importance of protecting our vital installations and people and how this can be done. But protecting the people is only part of the story. If they are to operate effectively they have to keep in touch with the outside world and with each other to learn what is happening, evaluate the situation, reach decisions and carry them out. To do this they must have communications, not only before, but during and after the attack.

What then are the prospects that they will have them? As things stand today, they are not good. This is not to contradict others who have pointed out that communications <u>can</u> be made very hard. But this, I want to emphasize, is a potential capability--something we can achieve only if we work hard to get it and are willing to pay the additional costs involved. It doesn't come from just saying we want it or from writing into specifications that we "require" it.

The reason for this is that the communications in this country were never designed with the thought that the U. S. might someday be a battlefield. But, in fact, this is precisely the kind of situation we face and we've got to take it seriously. Unless we do, the chances are that communications may turn out to be the weakest link in the chain. Perhaps this is one of the things that General Twining had in mind when he said recently that "communications is the weakest link in our military's capability."

Unfortunately, just recognizing the threat and taking it seriously aren't enough. People have known some of these things for years and I'm sure that most of them are obvious to you. Yet the problem is still very much with us; in fact, it is growing more serious all the time. Probably the main reason is that our thinking just hasn't kept up with the threat. We're busy today solving 1955 problems when what we should be doing is looking ahead to 1965 problems. Another reason, of course, is the fact that nobody gets very excited about communications so long as the installations they served are soft: after all, why worry about communications when there won't be anybody around to use them? But these aren't the only reasons why communications vulnerability is still a problem. The whole subject has been characterized by a lack of facts and a confusion of objectives. Even the terms we use in talking about it are subject to widely different interpretations, as you will see from some examples later.

But, whatever the reasons for not having made more progress on protecting our communications, the time has clearly come to take another look at the situation, what it means, and what we can do about it.

The first question we must ask ourselves is: What is the threat? This is easy to answer in a general sort of way, but to do anything constructive we must start with a realistic and objective view of the enemy capabilities, the probable patterns that his attack might take, and the effect on our communications. Unfortunately, many past studies have been neither particularly realistic nor objective. Some have tended to think of the attack in terms of what we would prefer to have happen instead of what the enemy is capable of; some have gone to the other extreme and imagined an attack so ferocious that the situation would be utterly hopeless. Neither of these extremes is likely to lead to constructive action.

In evaluating the threat we have to be clear what we are talking about.

Just what communications are we concerned about? Are we talking about having them survive the attack in the first place or restoring them later? Some past studies have been unclear on these points. For example: Some studies have stated results in terms of the total damage to all facilities, including the "general service"for the use of the public. These results are interesting but hardly meaningful to those whose networks are made up of private line circuits. The pertinent question here is: How does the attack affect each particular network? There has also been confusion between survival, which means the ability of communications to survive an attack, and restoral, which means the ability to reestablish communications if they are lost. Both are important, but clearly they have entirely different values to the user. Unless we are clear on such points as these our studies are unlikely to lead to constructive results.

Fortunately, some excellent work has been done recently which should help give a much clearer view of the nature and scope of the attack and its effects. The military services and the Department of Defense are evaluating enemy capabilities and probable patterns of attack. Other groups have studied the effects of weapons on various types of communications plant such as pole lines, radio towers, etc. Still other groups are setting up machine models to analyze the effects of various attacks on communications. All of these should contribute powerfully to a better understanding of just what we are up against.

After we have evaluated the threat, the next question we must ask is: what is the effect on our operations of losing communications? Understanding this relationship is absolutely essential to further action. It is easy to envision operations which would be affected only very slightly by a substantial loss of communications. It is also easy to visualize operations in which even a minor loss of communications could result in utter chaos. It is interesting to note that in either of these extreme cases there would be very little reason to concern ourselves about protecting communications. Either we would be so well off that there would be no point in it or we would be in such bad shape that nothing we could do would really help. Unfortunately, very little has been done to evaluate the interrelationship between communications capability and operational capability. By and large, operations analyses and exercises have failed to take the effects of communications loss into account. It cannot be emphasized too strongly that much greater effort is needed in this direction. Unless we understand this point it is unlikely that any constructive efforts to protect communications will be needed.

The third question we need to ask is : What can be done to reduce our vulnerability? It turns out we can do four things. We can build duplicate or 'back-up' facilities, we can harden the facilities we have, we can select our routes to avoid target areas, and we can diversify the circuits assigned to a particular network. No one of these solutions will solve the problem by itself; we have to look at all four and then decide the best combination of measures to meet the requirements for a particular system. The first two--building duplicate facilities and hardening-- are beyond the scope of this discussion. I will, therefore, confine the remainder of my remarks to the other two measures: the avoidance of target areas and the diversification of facilities. These measures deserve special attention for they provide the main method by which we can protect the bulk of our intercity long distance facilities. Furthermore they provide a promising way to achieve substantial reductions in vulnerability at a fairly moderate cost.

Before discussing these in detail I would like to take a moment to recognize the current efforts of the commerical communications companies. The Bell System and Western Union have been working along these lines for some time. These efforts are in the right direction and do credit to their wisdom and foresight. However, it is extremely unwise to assume that these efforts will solve the problem. The companies are bound by the terms of their service offerings to furnish service to all users at the lowest possible cost consistent with good service. They cannot engineer their plant to meet the special needs of individual users but can only provide what they feel is essential for all. The users who have critical requirements must work out the special arrangements needed to give them the further degree of protection which they require. They also must be willing to pay the extra costs associated with these special arrangements.

Now let's turn to the specific methods of avoiding targets, and diversification. What can be done by avoiding targets? The answer: a good deal. Our commercial communications facilities are numerous and widely dispersed. The trouble is, as the circuits are ordinarily ordered they do not take advantage of these diverse routes but instead are likely to follow main trunk routes. The result is that the "back door" routes which avoid targets represent only a restoral capability. What we need to do is to take advantage of these avoidance routes to increase our survival capability by specifying in advance that certain target areas are to be avoided.

Turning now to diversification; what does this accomplish? It simply spreads the risk as it applies to a particular system. Of course, even if our circuits avoid likely targets, some risk remains. The idea of diversification is to spread this risk as it applies to a particular system so that the loss of a single route or key point will not be fatal to the operation of the system. Most networks have many circuits, and while these are not completely interchangeable they can usually maintain essential service in a pinch. The details have to be worked out individually for each particular system since they depend in large measure on the kind of operation being conducted, where the circuits terminate, etc.

In closing I would like to emphasize two extremely important points. The first is that achieving improvement in communications vulnerability takes hard work. There have been a number of attempts in the past to solve this problem on a wholesale basis by establishing broad criteria applicable to all services. These attempts have almost always been disappointing. The plain fact is that to fully understand the effects of communications loss and to decide what to do about it we must take a careful look at each individual system, perhaps in some cases each individual circuit. This sounds tedious and it is. But we are convinced it is the only effective way to get at the problem. Fortunately a number of users with critical requirements have realized this and are beginning to work very effectively in this direction.

The second point I should like to emphasize is that it is extremely important for the users of the services and the companies to work closely together. There has been an unfortunate tendency in the past for each to assume that the other would take care of the problem. The users have often felt that it was up to the companies to solve the vulnerability problem; the companies on the other hand have felt that it was up to the users to specify what was needed. As a result nothing was done. Only the users can determine the need for protection; only the companies can determine how best to provide it and what it will cost. A solution can be reached only by their working together.

PROTECTION OF COMMUNICATIONS AND ELECTRONIC SYSTEMS

F.R. Eldridge
The RAND Corporation

We would like now to examine briefly some of the problems involved in providing protected communications and electronic facilities for super-hard sites of the types which are being considered in this symposium.

In general, the choice of a design for a communications and electronic system will depend on the feasibility of meeting certain operating requirements including the ability to protect the system against such things as physical damage, sabotage, radio jamming, etc. In considering physical damage, the designer will, in most cases, want to equalize the risk to the terminal sites and to the communications and electronics which serve them. In other words, it is a bad policy to leave any weak links in the overall system. Next, he will want to determine costs of the possible communications and electronic systems which he finds that he might be able to use. Finally, his choice of a specific system or a mixture of various types of systems would be made on the basis of feasibility as well as on the various types of vulnerabilities and the costs required to produce a system with the required protection.

As Jack Carne has indicated, most of the present landline and radio communications systems are very vulnerable to disruption by physical damage from enemy attacks. There are problems of weak antenna structures and overhead lines on telephone poles. Underground cable lines used in the present telephone system are brought above ground at intervals in weak facilities such as repeater points, central offices in large cities, and at river crossings. We may then be faced with a very difficult problem in trying to maintain communications after an attack.

What then can be done under these circumstances? Let us look for a moment at some of the types of hardened components which can be used for protecting communications and electronic systems. All evidence that we have, indicates that the cables can withstand very high overpressures.

A good example of modern cable design is shown in Fig. 1. Around a solid control conductor is extruded a thick layer of high-molecular-weight polyethylene with 5 per cent butyl rubber added. Over this a layer of copper tapes is wound to form a coaxial core. Five separate coatings are then applied to protect the coaxial core. The outside diameter is about 1-1/4 inches. A cable such as this is not only flexible but can and does take the tremendous static overpressures of 10,000 psi or more which exist in the deep parts of the ocean. Since cables, if buried underground, are not subject to dynamic loading of any significant amount, one might expect that they could survive very close to the crater of a nuclear burst.

Of course, certain precautions should be taken in installing cables to avoid breakage by earth's shear effects. Rock with possible shear fractures should be bypassed. Also, loops or pigtails may be used in the cable where it enters structures such as underground enclosures. Added protection may be gained by providing sufficient slack in the cable when it is laid. Furthermore, it can be covered by sand or other loose material before burying to give it further leeway for movement if bombs hit nearby.

Manholes must be provided at intervals in cable networks for servicing the signal repeaters, and for servicing and repairing the cables themselves. Figure 2 shows a typical design for a hardened manhole. It has been planned with the intent to protect electronic equipment used in the repeaters, such as transistors and capacitors, from high overpressures as well as from radiation damage which might occur in a bomb blast.

Means of protecting still other types of systems, which might provide critical communications during wartime, are also being examined. This includes a wide variety of possible communication techniques such as seismic, ultra-low frequency, microwave, scatter, satellite, and rocket communications, as well as aircraft relays. Many of these will be discussed in more detail in the classified sessions to be held tomorrow and the next day.

Some of these communication systems may require very large antenna structures. Figure 3 for instance, shows a possible design for a hardened troposcatter antenna, or perhaps a ground terminal antenna for a satellite relay communication system using orbiting chaff or dipoles. The horn feed from a wave guide would be supported by heavy I-beams. The parabolic reflecting dish, 30 ft. or more in diameter, would be constructed with steel plate 1/2 inch or more in thickness. The installation would be made by facing off a rock cliff and driving rock bolts into it. The antenna dish would then be raised into place and welded to the rock bolts. A grout backing would provide further support for the dish. It looks as though it may be possible to design this type of antenna to withstand as much as 200-psi static overpressures and equivalent dynamic pressures.

Another antenna design which looks even easier to harden is a matrix of slot antennas. It would be constructed from a number of slotted wave guides flush-mounted in concrete and filled with a dielectric material with high compressive strength. This design has other advantages. The direction of the electromagnetic beam can be varied using electronic scanning techniques. Therefore, possiblities exist for using it for hardened radars as well as for fixed-beam communication systems requiring large antenna gains.

Still other designs involve the use of retractable antennas stored in hardened silos with means for raising them above ground after an attack.

These, then, are some of the components which might be required for communications and electronic systems associated with a hardened control site.

In addition to hardening, we can use dispersal to protect weak points in a communications system. We refer here in particular to components such as antennas, which must necessarily be exposed to the atmosphere to function properly.

Consider, for instance, a case in which we plan to use, say, a hardened underground room capable of withstanding perhaps 1000-psi overpressures (Fig. 4). Suppose also we find that we can construct an underground cable hardened to several thousand psi which will be used to connect this control room to an antenna required for line-of-sight radio communications. However, suppose the antenna cannot be hardened sufficiently to protect it to the extent that the underground room is protected. As a result this antenna would be the weakest point in the complex. What can be done to improve matters? Under the conditions which have been outlined, a moderate amount of antenna multiplicity and dispersal offers a possible solution.

Suppose we were able to harden the antennas somewhat and to locate a number of them at great enough distances from each other so that they are independently vulnerable--that is, so that one nuclear weapon can knock out no more than one antenna.

By adding enough of these semi-hardened antennas, it is possible to make it as difficult to destroy <u>all</u> of these antennas as it is to destroy the hardened underground terminal site itself. For instance, it can be demonstrated that three antennas of only about 30-psi hardness, spaced at distances of 3 or 4 miles from a 1000-psi underground site, would have the same combined survival probability as the underground site itself (Fig. 5). This would be true regardless of whether bombs were aimed at the underground site or at the antennas. This, then, is a simple demonstration of the advantages of using mixed methods of protection.

These same principles can also be applied to protecting dispersed, semi-hardened landline communications. For instance, starting from a hard communication terminal room we may be able to provide a number of alternate routes for each critical circuit. The more independently vulnerable routes which are added for each circuit, the less protection will be required for each of the routes in order to maintain equal protection of the circuit and the terminal room.

On the other hand, in cases where a number of critical circuits emanate from a communications center, or where many critical circuits must be carried on a communications express route, it may be cheaper to maintain equal protection in the system by providing hardened circuits than by providing multiple routing. For instance, although an underground cable route, hardened to 1000 psi, might cost, say, three times as much as a microwave route of 2-psi hardness, the cable route would provide something like 30 times as much protection as the microwave route. In other words, any specific critical circuits would have to be carried on about 30 dispersed and independently vulnerable microwave routes of 2-psi hardness in order to gain the same protection as would be provided by the 1000-psi cable route. Conversely, on an equal risk basis we would probably be willing to carry 30 times as many critical circuits on the cable route as on the microwave route.

To summarize then, we see that, in order to protect vital communications circuits for hardened sites, we may have to resort to the use of many different types of communication systems which are independently vulnerable to the various methods which could be used to disrupt them. We will certainly require hardening as well as dispersal. By means such as these it appears feasible to meet the growing wartime threats to communications and electronic systems.

DISCUSSION

FROM THE FLOOR: Who makes the cable you showed in the first slide?

MR. ELDRIDGE: This cable is used for the transatlantic A.T. and T. circuits. It is manufactured by the Western Electric Company.

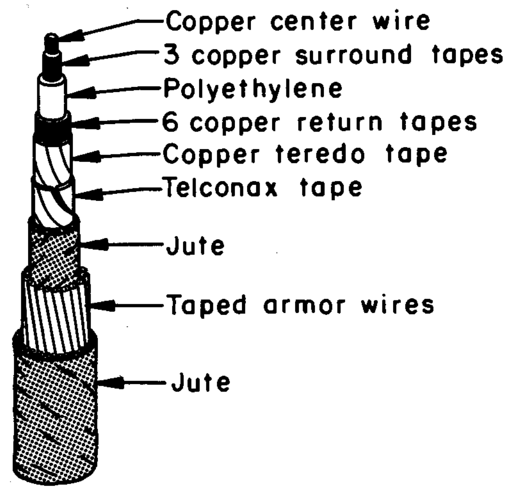

Fig. 1—Underground cable

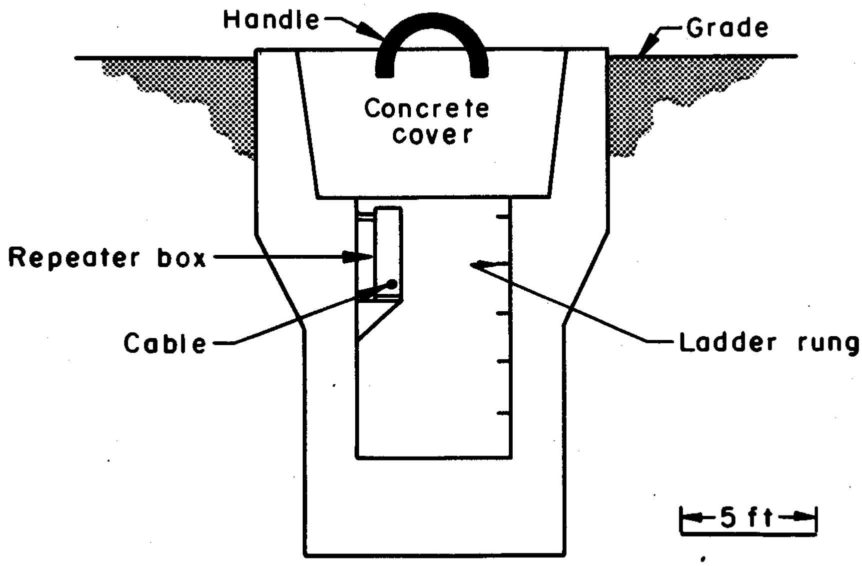

Fig. 2—Hardened manhole

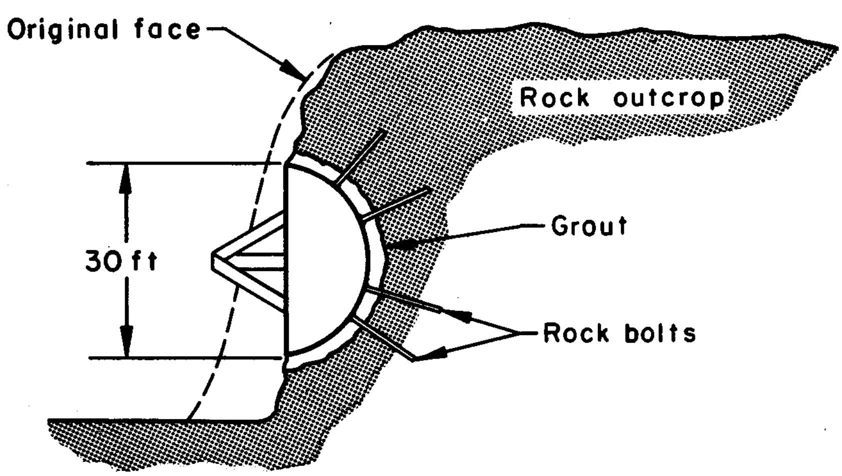

Fig. 3—Hardened antenna

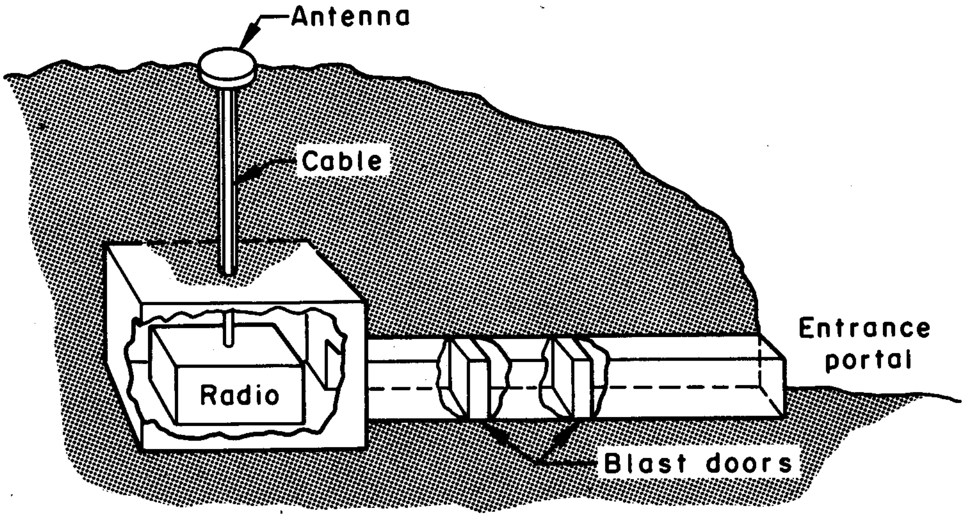

Fig. 4—Cutaway view of underground site

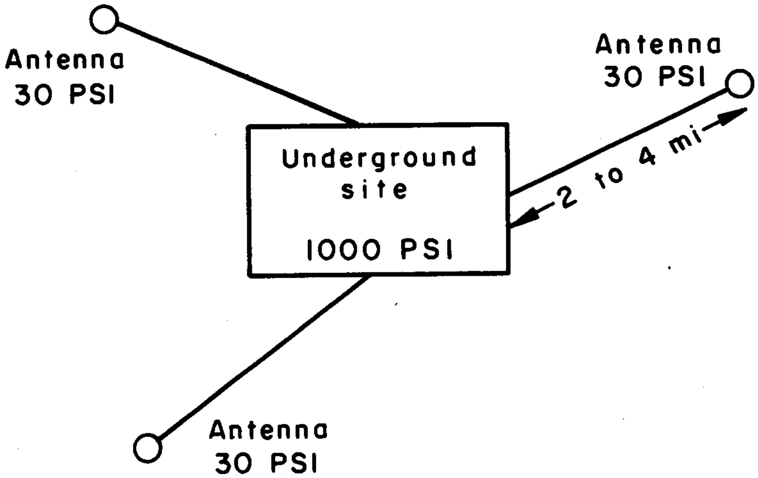

Fig. 5—Dispersal plan for semi-hard antennas

MINES AND OTHER CONCEPTS

Robert B. Panero
Guy B. Panero Engineers

The purpose of this presentation is to discuss some of the problems that face the man or the agency seeking protection and to describe some of the solutions available.

The person seeking protection is generally trained in management techniques and is rarely an operations research analyst, a scientist, engineer or architect. In all probability he has heard of mines. He thinks of them either as a panacea for all hardening problems or he may be scared from personal knowledge of mine disasters.

He has probably also heard of gaining protection through excavations in rock. Probably his knowledge of excavation methods or techniques is somewhat vague.

We will try to show some of the interactions between the various requirements of such a person (or agency) and develop a feel for the problem presented by high-quality protection in the nuclear and ICBM era.

We delivered a paper at the First RAND Protective Construction Symposium two years ago. This discussion presented some general thoughts on the availability and use of existing mine space. The paper was essentially designed to treat industrial exploitation of existing mine space. Since that time The RAND Corporation, as well as military and government agencies, have studied protective construction possibilities. We have taken part in many of these studies. Needless to say, our own concept, both of the problem and its solution, has changed materially since the First Symposium.

By way of introduction, we should establish that both the Corps of Engineers and the Bureau of Mines have investigated existing mine space in the United States, have catalogued such information and evaluated the suitability of various mine spaces. In general, this information is maintained up to date.

In addition, civil and military studies conducted by The RAND Corporation, which essentially treat the protection problem from the operations viewpoint, seem to point to the fact that underground space may be extremely valuable--might be considered a national asset.

The man or agency seeking protection is interested in both existing and new space underground, such as:

1. Basements
2. Tunnels
3. Vaults
4. Caves
5. Mines

In terms of new space, he may be interested in:

1. Semi-buried structures
2. Buried structures
3. Excavations in rock

This Symposium is interested in either existing space in rock or in new excavated space in rock, due to the high protective potential available in rock. We are concentrating on high-quality protection which is to be found at depths hundreds of feet below the surface in generally strong geological formations.

To discuss existing mines properly, we must first take a look at the mining industry as a whole. This industry extracts several billion tons of ore per year. The majority of the rock extracted by the mining industry results from surface operations, strip mines and quarries. Only a portion of the rock excavated by the mining industry results from tunneling operations. We are, of course, interested in the tunneling operations. We may well be interested also in quarrying operations insofar as such operations might be converted to tunneling operations by inducements of one sort or another.

Tunneling operations can be sub-divided into two categories:

1. High-price ore mining
2. Low-price ore mining

We will discuss each of these and will try to ascertain how suitable existing underground space develops, and the reasons for this development.

High-price ore mining would include gold mines which have a product that sells for $35 per ounce. The mine operator's job is to extract and sell this ore as economically as possible. His main interest is in maintaining a favorable ratio between marketable and unmarketable rock extracted. He searches continually for a pocket of high-grade ore in which all, or nearly all, the rock removed is marketable.

The very nature of the operation results in extremely irregular spaces and may well result in support problems in certain areas. Layouts of such mines will vary extremely in terms of room size, ceiling heights, and floor levels. An example of such extremes might be gold and silver mines in the Colorado area where small tunnels suddenly develop into enormous stopes, several hundreds of feet wide and several hundreds of feet high. These stopes are the spaces resulting from extraction of large pockets of high-grade ore.

The operator of such a mine, due to the high price of his product, can live with many conditions that other operators with more marginal mines would be unable to support. It is not surprising that this kind of mine does not result in space that is useful for the applications we are discussing here today. There are some exceptions to this statement--some lead-zinc mines in Oklahoma-Missouri--but it will generally hold true.

As to low-price ore mining, we can take the example of limestone mines in the Middle West. Here the product is merely crushed rock for roadbeds and concrete. This normally has a sale price which varies (depending on the area) from $1 to $2 a ton, or $2 to $5 a cubic yard. In this case the operation is generally marginal.

The mine operator literally cannot afford support problems or water problems. He is primarily interested in the extraction of fairly large volumes of rock at an extremely low cost. Since he cannot afford support in terms of timbering or metal shoring, he will build in large safety factors.

Space resulting from his excavations is normally regular in terms of room widths and ceiling heights. Floors are level and in general the mine is self-supporting. It is not surprising that this operation results in suitable space that can be considered and evaluated as to relative suitability or to a particular application.

However, we must realize that each mine operator has a different approach to the extraction of his ore.

Figure 1 indicates a floor layout of an existing limestone mine in the Missouri-Illinois area which shows pillar irregularities, varying separations between pillars--in sum, various types of extraction techniques.

This might be contrasted with Figure 2, which is taken from the floor plan of another limestone mine in the Ohio area. It demonstrates a regular excavation procedure with large volume of residual rock support, and this residual rock support is in a regular pattern. The space resulting is more suitable than that shown in the previous illustration.

The kind of space resulting from this mining technique results in areas similar to those shown in Figure 3, a photograph of a typical, average limestone mine.

It is obvious that the quality of the protection available in mine spaces is going to depend to a great degree on the depth of the mine and on the residual support left in the mine by the operator. Protection in many of these mines will vary from the low psi's--5 to 20, to 100 psi. Many mines might be found with characteristics in this protection range. A few mines will, however, be capable of providing protection of 100 to 1000 psi. The protection available in any given mine is going to depend somewhat on the application to which the mine is put. For example, even a low-cover mine, relatively close to the surface, may give a high degree of protection to a storage cache.

Before discussing the adaptation of mine space, let us take a look at some of the different criteria that might be established for civilian and military applications. We can assume that civil defense shelters would not be in themselves a target and that essentially low degress of protection may be suitable. In civilian use we can normally expect a large statistical advantage. That is, even if one factory is located underground, the dispersal of the surface industry is large.

In terms of civilian applications, the agency seeking protection will be interested in the anticipated effects due to the location of the mine site from the nearest prime target.

On the other hand, the military man, if he goes underground, knows that his installation is a target in itself. He is consequently searching for a high degree of protection and a capability of surviving a no-warning attack as well as an attack with multiple weapons. The military man will be interested essentially in operating units that must perform under attack.

We can best describe the difference between a civilian and a military application by discussing the exit problem. If a military unit survives and operates during an attack period, it is not as important that the personnel operating the unit be capable of exiting; military people can be considered expendable. The function of the underground military installation, then, is to operate during the attack period. On the other hand, the civilian application, whether for housing of persons or storing of goods, industrial tools, stockpiling of foodstuffs or critical material, fails if the personnel or material cannot be removed after the attack or the war is over.

We can see, therefore, that in the military case it may be permissible for entrances to close up and be destroyed under attack as long as the installation keeps operating. In the civilian case, some sort of exit is required. The problem is lessened by the fact that the civilian application is not a target, while the military is.

To summarize military versus civilian use, we can probably state that the military application--

- o will always be the sole occupant of the underground site;
- o must operate under attacks; and

- o is itself a target.

In the civilian application--

- o the use of a single site for multiple purposes is possible;
- o there will never be a requirement for operating under attacks; and
- o it will only have to survive and will not generally be a target.

(This is perhaps an over-simplification, but will generally hold true.)

USES OF MINE SPACE

Let us now discuss uses for the types of mine spaces we are discussing. You have heard of the problems of radio-active fallout as discussed earlier by Herman Kahn, as well as the importance of having a strategic evacuation capability.

In many cases, depending on location, such a capability is available in existing mines. Pittsburgh, for example, has approximately 100 million square feet of existing, readily accessible, suitable underground mine space within 50 miles of the downtown area. Other uses could certainly be warehousing or regional stockpiling. Warehousing or stockpiling in this case would be to survive the war and be available to "get the wheels turning" again. Such things as industrial stockpiles are being handled today in mines because this is more economical. Both petroleum and equipment are stored to a certain degree in underground facilities for this reason.

If we take a look at the suitability of mine space for the installation of an industrial plant, say a factory, there are many types of factories that could be installed at low cost in suitable mine space.

The industrial plant manager will be less interested in the construction economics than in the wage situation when he goes underground. The wages of his labor may be 10 times the construction costs. Consequently this may well be a marginal application for the industrialist because he does not know what the wage situation is.

Members of the RAND study teams have visited Sweden and discussed this point with Swedish manufacturers who do operate underground installations. They found no difference in the response of labor to working underground if the underground facilities were comparable to those above ground. We cannot assure that this holds true in the United States as well.

Another possible use is a civilian defense headquarters, possibly a regional civil defense headquarters--which could house stockpiles of food and medical supplies, possibly a radio station, and also double as a shelter. It would provide warning initially, and assist damaged areas after attack. All such applications seem feasible in existing mine spaces of the type discussed.

However, if we look at the problems associated with a military headquarters, or other possible military applications, the use of existing mine spaces does not seem practical. The criteria spelled out in terms of high degrees of protection, no-warning attacks, multiple weapons -- is too much to ask of a space that has been excavated for such an application by accident. It is possible that one or two mines might suit a military headquarters in terms of protection and operating locale. If this is so, it will be somewhat accidental.

CIVILIAN APPLICATIONS

How can you adapt existing mine space to some of the civilian applications discussed?

If we take the storage application, probably very little adaptation or new construction is required. Figure 4 is a photograph of an existing mine in the Kansas area, presently used as a commercial storage facility. You will note on this photograph that very little treatment has been given to the rock itself, and block walls have been used to separate the various storage areas. Other applications such as warehousing facilities, stockpiles of all kinds, and civilian defense headquarters can be located in a converted mine space with minimum treatment of this kind.

For example, Figure 5 shows minimum adaptation of a mine in Missouri where the owner has constructed a garage and his own office below ground. You will note the finish is again minimum with use of block walls, exposed rock surfaces, lights and conduits attached directly to the rock.

Figure 6 is a photograph of the interior of the mine operator's office, which again is an example of minimum finish. Walls are directly supported by the rock, as is the ceiling. It is evident that, given the space, it is quite possible to put in such modifications for very low per-square-foot costs. Obviously, economies can result from this kind of adaptation. Where such economies are sufficient to outweigh the cost of construction on the surface, you will find industry applying this approach.

Where is such existing mine space located? Figure 7 indicates an area of the United States where the majority of this type of space is located. There are some exceptions. Generally the area shown here represents the limestone belt of the country and is dotted with limestone tunneling operations and limestone quarries.

In terms of suitable space, there are probably more than 400 existing suitable mine sites representing more than 600 million square feet of space. At least 100 million square feet of the space available is in perhaps 10 enormous sites. This large amount of floor space existing should not be confused with usable square feet resulting from adaptation. This space increases at the rate of approximately 50 million square feet per year. This could probably be increased by various federal or other inducements by a factor of 2 to 5. It might be possible to order custom-tailored space cut out with higher safety factors than normal and in regular usable increments.

How do you go out and acquire a mine? Given the light distribution of mine sites and the varying suitability of mine sites in general, it is relatively easy to decide that only a single site can serve the purpose in question. This is a pitfall that should be avoided.

The viewpoint of the mine owner is that his space is really a subsurface structure, that he will have to halt his operation, and he can no longer count as part of his capital the residual rock that he has in the space for support. He will try to sell the mine on a square-foot basis and may well ask for something in the neighborhood of 50¢ to $1 per square foot. He will not normally want to rent the space.

The viewpoint of the purchaser of the space will be that he needs 1/10th of the mine space available and that the mine operator has, for all practical purposes, abandoned that space. Consequently the purchaser wants the space for nothing, the mine owner wants the highest price possible. A solution may be for the purchaser to get bids from two or more mine owners. Comparing two or more existing mine spaces is very difficult. They are never identical. They vary in terms of protection and suitability. They vary in terms of location vis-a-vis the operating locale. In all cases the purchaser must balance the economies involved by use of an existing mine against the cost of excavating a site tailormade to his needs.

It is worth noting that the civilian is in a much better position to negotiate for such property than is the military. The military is limited by the rules and regulations pertaining to such purchases. The civilian, on the other hand, has more negotiation freedom.

Mine acquisition at reasonable cost is difficult and requires some advance planning.

MILITARY PROTECTION

Let us establish how the protection requirement develops and discuss the various factors that influence the man or agency seeking protection. In particular, let us discuss this from the point of view of the military headquarters.

Essentially the requirement for protection results from a calculated operational need developed by an assessment of many factors and prepared by operations analysts. The analyst realizes the magnitude of the errors possible in his calculations. He therefore would like to impose on the designers certain features which may well be essential to the proper solution. More often than not he never meets the designer.

The first feature of interest is flexibility--to design into the installation a capability of accepting changes in operational criteria at a future date. The second feature desired is a capability for retro-fitting such installations--to so design the facility that modifications, variations, and expansion of the facility can take place at a reasonable cost and in a practical manner at any time in the future. The third item, which may well be the most essential of the three, is a need for speed. Generally the calculations made by the operations analyst, although in themselves not establishing the requirement for speed, may be in error. The need for increasing construction rates may become acute at any time. Consequently, design should be such that it can shift from low-gear or normal construction procedures to high-gear, high-speed construction methods.

Now, all of us know, particularly the engineers, that this sort of requirement results in continual changes and continual delays. It is extremely difficult to complete any design on this basis, particularly on time. It discourages the engineer, the designer, the draftsman. However, it is something we must learn to live with. We cannot confuse design difficulties with operational unusability. In comparison, the design difficulties are cheap. The fact that engineers may "tear their hair out" over continuing changes can never counterbalance the fact that installations, prior to completion of construction, may be unusable for lack of design change. The unusability of facilities is due to rapid changes in weapons technology, as has been pointed out by Herman Kahn in an earlier discussion. A friend of mine at OCDM once told me, "Every time you think you have the problem licked the bomb gets bigger or it gets here faster." His statement referred to an attempt to freeze shelter designs.

It may well be necessary to increase engineering efforts for this type of work and higher fees may be required. Mr. O'Sullivan in his paper mentioned the Naval Architect's approach, which is a very fine one. We do not know at this point how their fees compare with the usual Architect-Engineer-fees.

ENGINEERING DESIGN PROBLEMS

Let us now discuss some of the design problems that face the engineer trying to solve the protective problem presented by an agency. The most important thing that the designer must understand is that he should design to eliminate problems wherever possible, rather than to alleviate the conditions that are present with any given problem. For example, when you go underground you have, to a certain degree, eliminated many of the surface conditions which present problems to the designer of buildings at or near the surface. You will hear discussed at this Protective Construction Symposium some of the closure problems and how these problems can be eliminated by, for example, oxygen generating and carbon dioxide absorption equipment stored within the installation. You will hear discussions on heat sinks and nuclear reactors--all of which, to a certain degree, eliminate problems rather than alleviate them.

Given a decision to harden a headquarters-type installation, there are many factors to be considered. Here is a tabulation of some of them. They are subdivided essentially into operational and vulnerability factors:

1. OPERATIONAL FACTORS
   - A. TYPE OF INSTALLATION
   - B. OPERATIONAL LOCALE
   - C. PROTECTION REQUIREMENT
2. VULNERABILITY
   - A. DEPTH
   - B. CONNECTIONS TO SURFACE
   - C. OTHERS

Certain items are of particular importance at this point. The development of the protection requirement (Item C under Operational Factors) is of primary importance. It is here that the Operations people measure their basic protective situation. They must balance such things as the threat the enemy is capable of putting on their operation. They must measure this enemy capability and the importance of their own installation to the overall military situation. Against this must be balanced the statistical advantage represented by other similar installations or similar systems, protected or unprotected. After calculating things of this kind they must look at the budgets or funds available to them, and particularly in this case they must weigh the political factors and try to find some sort of compromise between the ultimate operational system and a practicable, sellable solution.

It is easy to see that in developing such an Operations calculation, large errors are possible. Prior to making a final decision, system costs must be available to them for comparison. Consequently, they must have a feel for the cost variance between using existing underground space or developing new underground space for their particular situation.

Before continuing with a discussion of how such decisions can be made, we should like to establish some of the advantages and disadvantages that are present in comparisons between existing underground space and newly excavated underground space. Certain trade-offs are required; consequently these advantages and disadvantages should be clearly in mind when evaluating the military headquarters protection question.

<u>Existing Mines</u>

<u>Advantages</u>

1. High-speed installation of facility.

2. Reduced costs possible.

3. Generally, considerable excess space available.

Disadvantages

1. Fixed depth.

2. Fixed internal support.

3. Consequently--fixed protection capability.

4. Fixed location.

5. Generally--inefficient space available.

6. Acquisition difficulties.

7. Light distribution of possible sites.

New Sites

Advantages

1. More efficient space.

2. Increased depth and protection available.

3. Increased support can be provided.

4. Many possible locations.

Disadvantages

1. Can be extremely slow, depending on type of installation and design.

2. Can be extremely expensive, depending on type of installation and design.

With these items in mind, the agency seeking protection can proceed with the evaluation of the factors indicated on the outline we are considering here.

Let's now look at Item A under Vulnerability.

In comparing a fixed mine location with a new site, many trade-offs are possible. It is possible to compare an existing mine with 200 ft of cover to a new site in roughly the same location in which certain sacrifices have been made in the space excavated to increase the depth and consequently the protection of the installation. In a mine we have a fixed-pillar situation. In a new site we can increase the residual rock support to any degree compatible with the operational requirement. An example of such trade-offs might be to use extremely low-cost interior housings in a fixed budget problem and apply most of the budget to increasing tunnel length, in consequence using the available budget to achieve the highest quality protection possible.

As a result of some of the studies we have participated in, we do have a roughly-outlined position on this sort of thing. Normally on trade-offs of this kind we would work somewhat backwards from a budget point, employing minimum solutions wherever possible and applying the maximum amount of the funds available to either increasing the depth of the installation or to reducing the vulnerability of the essential connections to surface.

We feel that the last item to freeze in terms of design is the access tunnel lengths, since this is the irrevocable decision. We can always reinforce a structure inside a tunnel at a later date. We would have to abandon it if we required additional depth. There are three kinds of access to such an installation. In all three cases the depth of the installation is related to the access tunnel length. The three kinds of entry are:

1. Shaft
2. Inclined
3. Horizontal

In the case of the shaft entrance we have the most efficient situation where by increasing the shaft length by one foot we increase the depth of cover of the installation as a whole by one foot. Inclined access tunnels would be somewhat less efficient and probably the ratio of increased depth of cover to increased tunnel length would be determined by the maximum slope vehicles can employ. Probably this would not exceed a ratio of 1 to 4. The ratio in the case of horizontal access tunnels depends on the general topography. We would not normally expect more than a 1 to 3 ratio.

Average engineering cost for these different types of entries for mixed formations varies from $1000 per foot for the shaft, to $800 for the inclined, and $400 to $600 for the horizontal tunnels.

It is important to note that with the limited knowledge available at this time on the effect of nuclear weapons on deep underground cavities, we employ this particular approach to try to maintain a flexible design, permitting some retrofitting at a later date. We consequently emphasize the importance of getting as much depth as possible now and as little dependence as possible on connections to the surface. We are not suggesting extreme depths or unlimited use of a calculation that equates everything against depth. We are talking in terms of buying additional depth in quantities of hundreds of feet by means of increased tunnel length.

To summarize our approach, although many factors relate to the vulnerability of an installation, the two most important appear to be (1) the depth, and (2) the vulnerability of connections to the surface.

In Item 2 C, we mentioned "other" factors. These would include such things as tunnel spans, residual rock, shock mounting, interior housings, and type and shape of tunnel cross-section.

All of these factors must be understood, costed, and analyzed with a view to overall installation hardness.

We have discussed some of the possibilities of existing mines. We have discussed some of the choices available to the agency requiring underground protection and we have attempted to go into the problem, in a minor way, of the modern control center. This type of control center is an extremely special case. It actually is not a facility; it is rather a control system with certain system requirements.

Here is another outline on which we have attempted to spell out some of the solutions available when considering the need for a protected control system.

PROBLEM
PROTECTED CD CENTER FUNCTION

Some Operational Solutions	Some Communications Solutions	Some Construction Solutions
Dispersal	Multiplicity	Surface Building
Mobile System	Local Protection	Basement Structure
Hard System	Network Protection	Existing Underground Space
Funds Available	Radio Backup	Excavated New Site

You will note that there are three categories of solutions available, falling in these areas--

1) Operational

2) Communications

3) Construction

If you analyze each of the solutions available you will note that a solution in one area has a direct effect upon a solution in another. For example, a decision to use a mobile system to achieve the degree of protection required would materially affect both the communication and construction problems. It is a combination of solutions in these three categories that results in a hard system.

If you carry this further and attempt to list all the various decisions required to achieve an operational date, you will see that this sort of analysis and the interacting effect by the three areas of interest spelled out here, run through all the decision-making prior to facility design and most of the decision-making during the facility design.

As an example, consider a typical decision for a hard operational system of the fixed base type in a new site. If we discuss the siting factors of such a system decision, we will see that the operator, the communicator and construction engineer are continually forced to rely on one another prior to making a decision. The reasons for this are fairly evident. Where system design involves an operational item, in many ways this type of system design compares directly to that required for aircraft development or surface vessel development, as pointed out by Mr. O'Sullivan in his discussion.

The interaction extends down to individual design increments. This can best be pointed out by the fact that one of the engineering papers to be delivered at the Symposium is devoted to the interaction of utilities in deep underground installations. In the discussion of the interaction of utilities themselves it should be realized that most of the assumptions made for any utilities system will be determined by operational and communications requirements. It is not surprising that a fixed or frozen decision in any single area of interest can materially affect costs in other areas of interest. There appears to be a need in the case of the protected military headquarters problem for an integrated team covering the fields of communications, electronics, construction, and operations research.

We might well conclude that in this sort of system development, the operations analyst is less effective without an electronics engineer and a construction engineer at his side. The electronics and construction engineers will be as ineffective without corresponding assistance or complete understanding of the operational problem.

PROTECTED CD CENTER FUNCTION
TYPICAL DECISION

Operational Decision	Communications Decision	Construction Decision
1. Hard System	1. Local Protection	1. New Site
2. Remote Location	2. Multiple Protection	2. High Degree of Protection
3. Operate as Independent Unit	3. Soft Network	

In conclusion, we should point out that inadvertently the construction design engineer can price this type of installation out of business. This does not mean that the problem disappears. It means that the operations analyst directs his studies toward other areas. If, due to high costs, he is unable for political or budgetary reasons to employ a fixed base solution, he may well decide on mobile, semi-hardened control systems to solve his problem.

Regardless of the viewpoint which establishes your interest in underground installations of any kind, and regardless of whether this interest is in existing or new space, or in military or civilian applications, we feel that it should be acknowledged that such installations do not represent a classical engineering problem. Although many choices are possible and many alternates can be presented, we must realize that this is a new problem. It is a problem peculiar to our age where rapid advances of weapons technology have obsoleted facility after facility prior to or at completion. We can only hope that the tools and mechanics can be developed to solve the problems posed by military and civilian operations in the ICBM era and that such problems will be solved in time.

FIGURES

Fig. 1

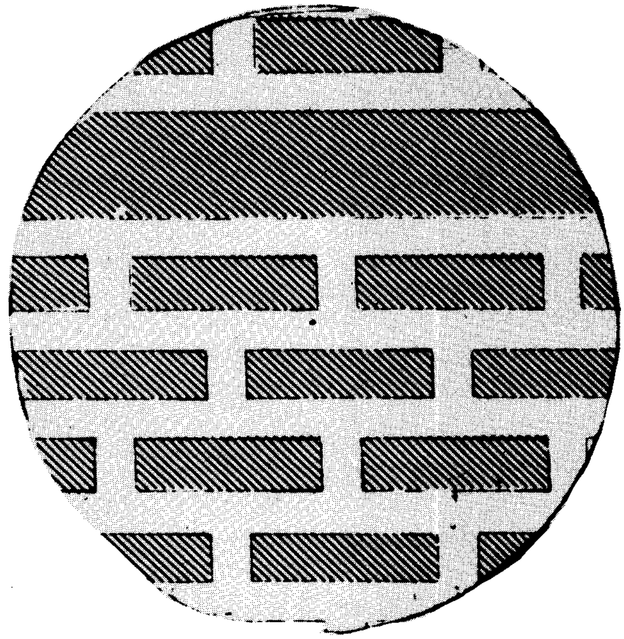

Fig. 2

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Fig. 7

SPALLING AND LARGE BLASTS

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INTRODUCTION

Large contained nuclear blasts impart tremendous forces to their immediate surroundings and these forces, becoming distributed in accordance with well-known laws of physics, product damage in areas both near and remote from the blast. Many qualities of damage manifest themselves, and of these, one of the most esoteric is spalling -- separation of rock -- frequently close to a free surface, as a consequence of preferential partitioning of the momentum of the stress wave. The elemental principles of the dynamics and mechanics of spalling in soils and rocks are discussed here. These principles are then used to appreciate more fully the significance of the observations of ground movement at the Rainier blast.

PROPAGATION OF TRANSIENT DISTURBANCES

When a buried nuclear bomb explodes, the action of the blast is not instantaneously transmitted to regions remote from the blast. Changing stress situations will be communicated through most materials at velocities of several thousand ft/sec, the exact value depending upon the material, the type of stress distribution, the state of stress within the medium, and the various boundaries involved. In large earth masses, velocities of two general types are encountered:

The dilatational velocity, c_{ℓ}, the velocity of propagation of longitudinal waves in a medium of infinite extent, is given by

$$c_\ell = \sqrt{\frac{3K(1-\nu)}{\rho(1+\nu)}}$$

where K is the bulk modulus; ρ is the density of the material; and ν is its Poisson's ratio. Variations of K, ρ, and ν with stress affect the value of c_ℓ.

Shearing displacements advance through a material with a velocity given by c_t

$$c_t = \sqrt{\frac{G}{\rho}}$$

where G is the rigidity modulus.

If the stress level is exceedingly high, the material will behave as a fluid, having no rigidity, and transverse or shear waves cannot exist within it. Under these conditions, stress changes will be propagated with a velocity, c_f, given by

$$c_f = \sqrt{\frac{K}{\rho}}$$

The state of stress will affect both K and ρ. Indeed, at the exceedingly high pressures produced by nuclear blasts, changes in values of K and ρ can amount to 30 or 40 per cent.

A stress disturbance passes through a body in the form of a transient wave of particle motions. A point or infinitesimally small particle within the body will be subjected to forces which impart transitory motions to it. The instantaneous velocity, v, of such a particle is directly related to the instantaneous stress σ at that point by the equation $\sigma = \rho c v$

where c is the velocity of propagation, viz., c_l, c_t, or c_f, of the disturbance. Particle velocity in a longitudinal wave is parallel to the direction of propagation of the wave, moving along with an advancing compressional wave and opposite to an advancing tension wave. Particle motion within a transverse wave is normal to the direction of propagation of the wave. In a typical granite, a particle velocity of 3 ft/sec corresponds roughly to a stress of 1,000 lb/in^2.

A typical disturbance, such as might have been generated by an underground nuclear blast, is shown in Fig. 1. The particular parameters used in plotting the curve are selected for convenience. Note that there is a one-to-one correspondence between stress and particle velocity. Whether time or distance is used as the abscissa will depend upon whether the phenomenon is viewed at all points in space simultaneously or at a single point through which the disturbance is passing. The instantaneous velocity of propagation of any point on the wave depends upon the level of stress at that point, being given by the expression

$$c = \sqrt{\left(\frac{d\sigma}{d\epsilon}\right)\Big/\rho}$$

where $(d\sigma/d\epsilon)$ is the slope of the stress-strain curve at that stress level. Unless the stress-strain curve is linear, which is seldom the case, the different points on the disturbance will range in velocities. Thus the shape of the disturbance will be continually changing as it moves along. In the particular case shown in Fig. 1, the wave would tend to steepen its front since the higher stresses would be traveling at greater velocities than the lower stresses.

Rocks exhibit a wide variety of stress-strain curves. Typical curves for limestone and sandstone are shown in Fig. 2. Note that the limestone curve is concave downward, which means high stresses will travel at relatively low velocities. The sandstone is concave upward, implying that the high stresses will travel at high velocities. In limestone, the waves will tend to elongate and flatten out, with just the reverse being true in sandstone.

DYNAMICS OF SPALLING

Viewed simply, a material is said to spall when one segment of it parts and moves off from another segment. This will occur if the velocity of the one segment is greater than the velocity of the other, as illustrated in Fig. 3. Stress waves, on reaching a free surface and reflecting, are prone to give rise to this type of partition of velocity or momentum. Consider the semi-infinite body of Fig. 4, in which a stress wave of the shape shown is advancing toward a free surface. Assume, further, that the stress-strain curve is linear. When a disturbance strikes the free surface, it will be reflected as a tension wave without change in form. The incident compression wave and its reflected tension counterpart will interfere with each other as indicated. The resultant distribution of stress within the body at a slightly later instant is shown in the right-hand drawing, the tension, AB, increasing as the reflected wave moves to the left. At some point, the material will no longer be able to support this tension; it will fracture; and a spall will fly off, trapping much of the momentum of the wave.

The stress required to rupture the material is called its critical normal fracture stress, σ_c. In metals, this fracture stress is exceedingly high, corresponding to differential particle velocities of 100 to

200 ft/sec. In rocks, the stress is quite low, being in the neighborhood of 500 lb/in^2 (1 to 2 ft/sec particle velocity for granite). The critical normal fracture stress is an extremely important mechanical property of the material and is the factor which governs parting of the material. Note that two factors are important in describing spalling: the shape of the stress wave, and the critical normal fracture stress. Determination of the thickness of the spall is a simple geometrical problem, provided σ_c and the shape of the wave are known. Thickness of spall will be equal to $\frac{1}{2}$ the distance within the wave that corresponds to a decrease in stress equal to σ_c.

Multiple spalling occurs when the stress level within the wave is more than double the critical normal fracture stress of the material. In such cases, several juxtaposed and parallel spalls are generated, as illustrated in Fig. 5. These arise in the following way: assume that the first spall is generated in the manner just described. The remainder of the wave will suddenly find itself impinging on a freshly created, free boundary surface. The result will be the formation of another spall. This process will repeat itself until the stress level of the wave has been reduced to a value less than σ_c. In the case illustrated, the thicknesses of the two spalls are vastly different. The first spall is quite thick because the leading portion of the wave is relatively flat. The subsequent abrupt decrease in stress thins down the second spall.

As pointed out earlier, underground blasts can give rise to disturbances of a wide variety of shapes which continually change as they propagate. Four idealized shapes are illustrated in Fig. 6. In view of the strong dependence that the thickness of the spall has upon shape of disturbance, it is apparent that each particular shape of disturbance generates its own characteristic spalls. Given the shape of the wave and the fracture stress of the material, it is, of course, possible to describe precisely the character of these spalls. The simplest shape to consider is the sawtooth curve in the upper left-hand corner of Fig. 6. This shape does not differ appreciably from that of the disturbance produced in the consolidated tuff 900 ft from the Rainier shot. (See Fig. 7.)

INFLUENCE OF MATERIAL PROPERTIES ON SPALLS

The tensile strengths, compressive strengths, competencies, and anisotropies in mechanical properties range widely in rocks and soils. The foregoing section described spalling for the very particular situation where the rock is perfectly elastic and isotropic. It was also tacitly implied that the material could support a compressive stress greater in magnitude than the tensile stress it could support. In real situations, such an ideal circumstance will not obtain. Compare now the three differing situations shown in Figs. 8, 9, and 11.

The first of these, Fig. 8, illustrates the spalling which will occur when a sawtooth wave, of length λ and maximum stress σ_0, strikes the free surface of an elastic material having a critical normal fracture stress σ_c, which is between 1/2 and 1/3 of σ_0. Two spalls are thrown off. Both are of equal thickness, δ given by

$$\delta = \frac{\sigma_c}{2\sigma_0} \lambda$$

but of different velocities, the piece to the right having the highest velocity. Specifically, the two velocities, v_1 and v_2, are

$$v_1 = 2v_0 - \frac{\sigma_c}{\rho c} \quad ; \qquad v_2 = 2v_0 - 3\frac{\sigma_c}{\rho c}$$

As the detached spalls move to the right they will become separated by an amount which is easily calculable using the above equations.

Figure 9 illustrates what will happen under similar circumstances when the material is incompetent, incapable of supporting any tension whatsoever. It is assumed, however, that a compressional wave will be transmitted without change in form. As soon as the wave reaches the free surface, the material will begin to flake off. The velocities of these first flakes will be equal to $2v_0$ where v_0 is given by

$$v_0 = \frac{\sigma_0}{\rho c}$$

Each bit of flaking absorbs a small portion of the momentum of the incident wave. The remaining portion of the incident wave will continue to advance against the freshly-created free surface, flaking continuing so long as any portion of the wave remains. The velocity, v_p, of any small particle will be given by

$$v_p = \frac{\sigma}{\rho c}$$

where σ is the value of the stress at the front of the incident wave at the time the particle leaves the surface. The initial thickness of the flaked-off material will be $1/2\,\lambda$. The flakes themselves will appear as a cloud of debris, the rear portion of which is stationary and the front moving forward with a velocity $2v_0$. Thus the extent of the cloud of debris is continually increasing, its forward portion becoming less dense.

The situation for incompetent rock or soil is examined in a little more detail in Fig. 10. Here are compared the displacements of points within a competent, elastic solid which does not spall and an incompetent mass which flakes off. Note that in the competent solid, the two points, A and B, eventually undergo the same displacement but do not do so simultaneously. The reason their total displacements are the same is simply that the material does not fly apart, and such must therefore be the case. The incompetent material behaves differently. Point A initially acquires a velocity which is maintained so that A moves farther and farther away from C. B moves along at a somewhat lower velocity. If the free surface of such a mass were horizontal, the restraining force of gravity would affect the material so that it would be brought to rest. These two drawings illustrate graphically the primary reason why damage is so much greater in unconsolidated soils and unconsolidated material than in solid rock.

The action of the wave on a series of juxtaposed slabs is illustrated in Fig. 11. The joints between the slabs are assumed to be perfect transmitters of compression but possessing no strength in tension. Each slab is of thickness L. On striking the free surface, the incident wave will be reflected, with the reflected portion proceeding backward (into the left) into the rightmost slab. When the wave reaches the first joint, the rightmost slab will fly off, trapping in it the momentum in the wave back to a distance 2L. The velocity of this slab, v_{s1}, will be

$$v_{s1} = v_0 + v_1$$

The second slab will then be carried off in a similar fashion with a velocity given by

$$v_{S2} = v_1 + v_2$$

and finally, the third slab with a velocity

$$v_{S3} = v_2 + v_3$$

THE RAINIER SHOT

It was observed in the Rainier shot that one or more large slabs of consolidated tuff, or rhyolite, situated about 900 ft above the blast, as illustrated in Fig. 12, rose to a height of 9 in. and then fell. From these data and accelerometers placed in a vertical hole situated directly above the blast, it was possible to make a reasonable guess of the shape of the stress wave which reached this top layer of rock. The most probable curve is shown in Fig. 7. The leading portion of this curve approximates the sawtooth waves whose actions have been discussed above. In Fig. 13, the wave is redrawn as a stress vs distance plot and shown approaching the free surface of the rhyolite. The specific gravity of the rhyolite was taken as 2.7. The maximum stress in the wave is 1,500 lb/in^2, corresponding to a maximum particle velocity of 7 ft/sec, if we take an average velocity for propagation of a wave of 6,000 ft/sec, close to the measured value. Such a wave would produce three spalls of equal thicknesses in a material having a tensile strength of 500 lb/in^2, a value reasonably close to that which might be expected for rock of this type. The three spalls would move away with initial velocities, 5.8 ft/sec, 3.5 ft/sec, and 1.5 ft/sec.

Gravity would slow these slabs down so that the total excursion of the top slab would be 7 to 9 in. Taking a somewhat lower tensile strength, say 250 lb/in^2, several more, but thinner, slabs would be generated. The thickness of the consolidated tuff layer was 230 ft. It seems likely that the entire slab probably moved as a unit with additional spalling taking place in the material below. All of the observations made are entirely consistent with what can be predicted from an appreciation of the elements of spalling. The two most important parameters needed are the shape of the wave and the fracture strength of the rock. In most situations, the rock will be stratified so that it will come off in layers of thicknesses preselected by Nature.

In considering the possibility of damage, it should be noted that the effect of gravity restricts substantially the movement of rock, primarily because the velocities involved are so low. Spalling produced on the walls and roofs of tunnels and other underground openings will not have the mollifying effects of gravity.

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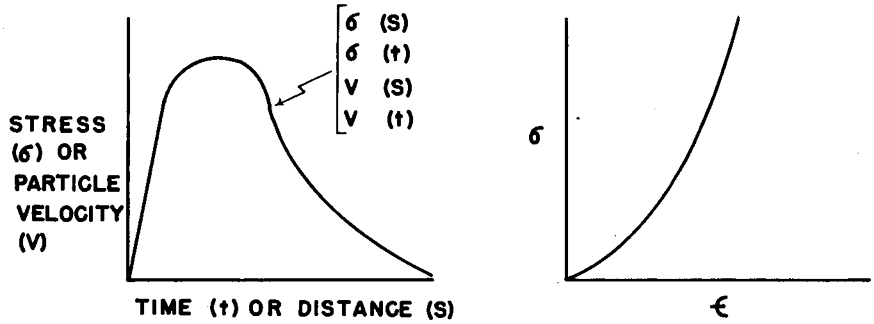
DISCUSSION

MR. CARL L. MONISMITH (University of California, Berkeley, California): How were the stress waves which you showed for limestone and sandstone determined?

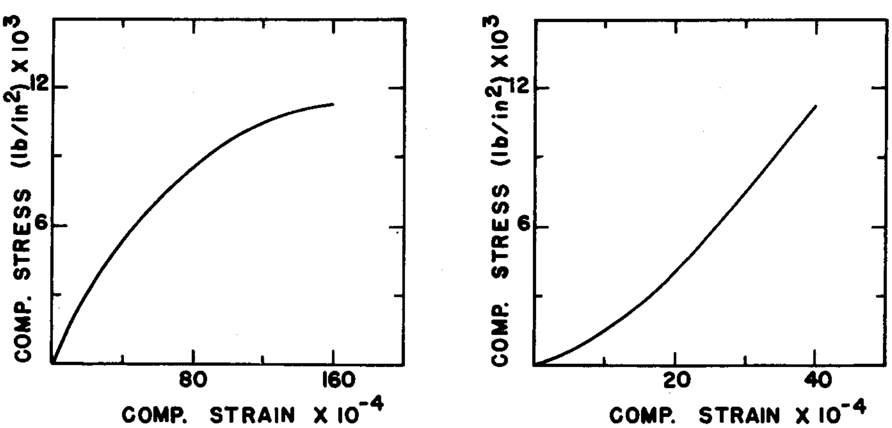
PROFESSOR RINEHART: Those were determined by the Bureau of Mines, and they were determined statically. There have been, of course, a great deal of data obtained by various people. I think perhaps the most active worker in this field as far as obtaining good quantitative data is concerned on the way in which stress waves behave as they travel through rock is Dr. Duvall of the Bureau of Mines.



$\sigma = \rho C V \qquad C = \sqrt{\frac{d\sigma}{d\epsilon}/\rho} \qquad \sigma = \sqrt{\rho \frac{d\sigma}{d\epsilon}} \cdot V$

C IS VELOCITY OF POINT ON WAVE

FIG. I — TYPICAL UNDERGROUND DISTURBANCE



DOLOMITIC LIMESTONE
(GREEN RIVER)

SANDSTONE
(NAVAJO)

FIG. 2 — STRESS-STRAIN CURVES FOR TYPICAL ROCKS

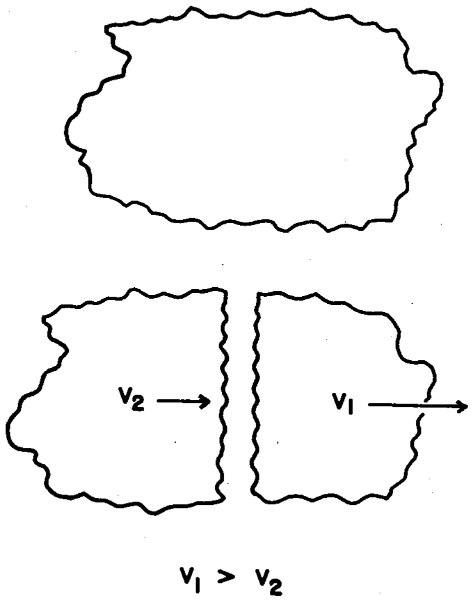


FIG. 3—DEFINITION OF SPALLING

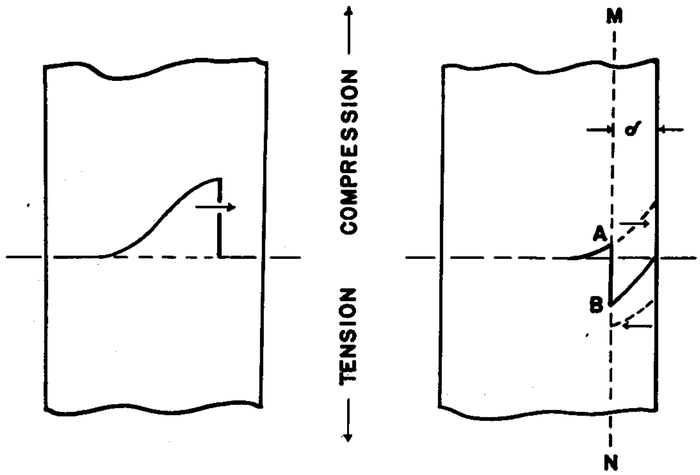


FIG. 4—STRESS WAVE IN A SEMI-INFINITE BODY

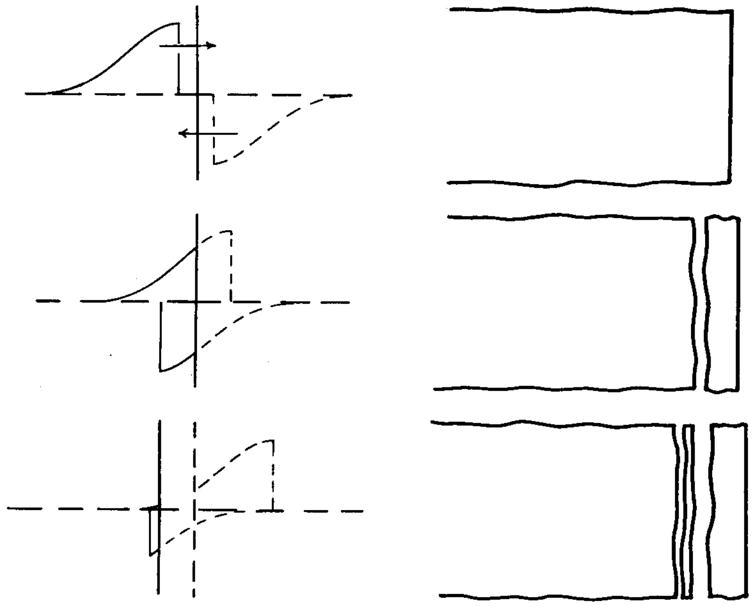


FIG. 5 —— MULTIPLE SPALLING

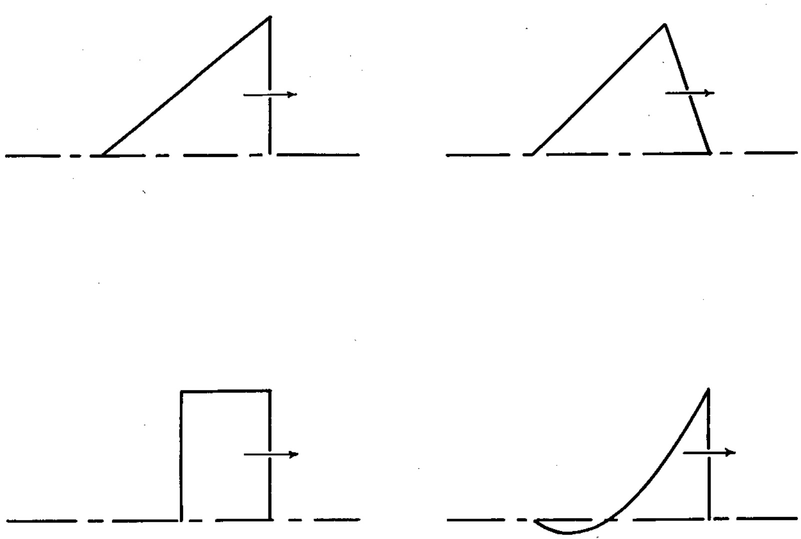


FIG. 6 —— IDEALIZED DISTURBANCE SHAPES

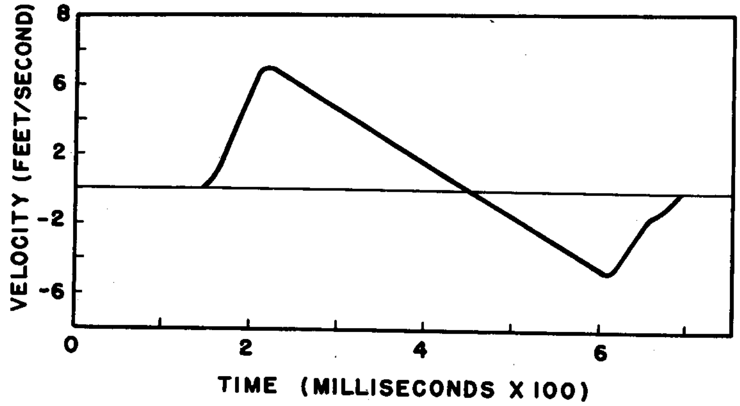


FIG. 7 —— DISTURBANCE IN CONSOLIDATED TUFF — RANIER SHOT

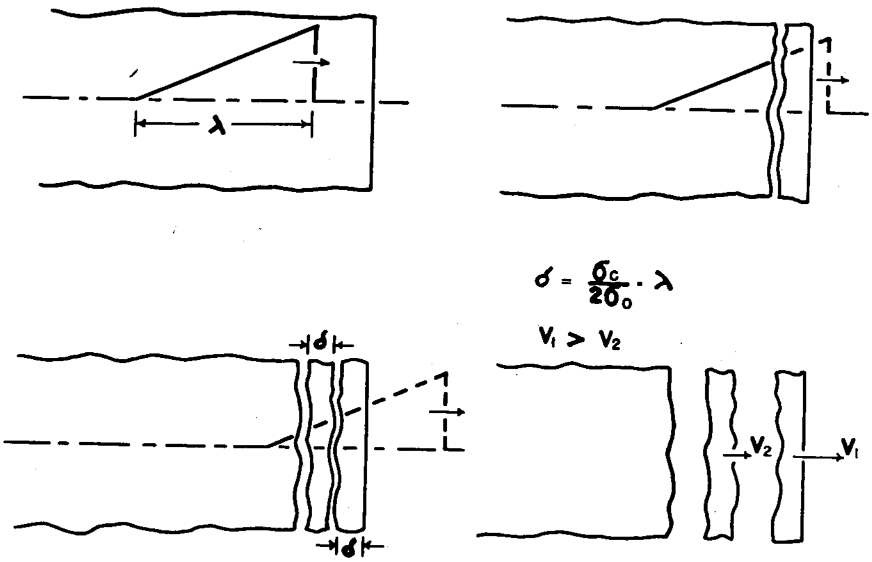


FIG. 8 —— SPALLING FROM SAWTOOTH WAVE

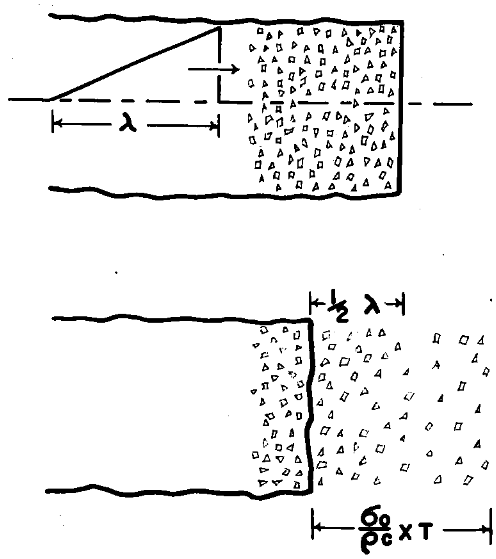


FIG. 9—FLAKING IN INCOMPETENT MATERIAL

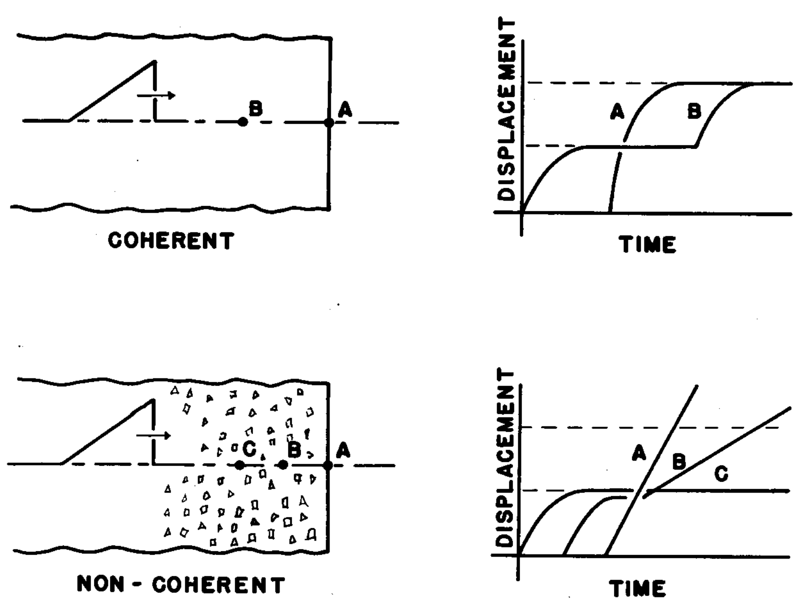


FIG. 10—WAVES IN COMPETENT AND INCOMPETENT MATERIAL

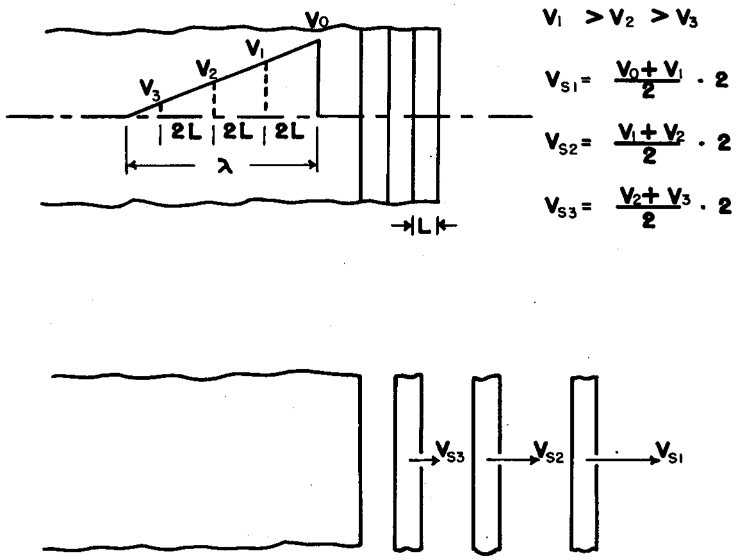


FIG. 11——WAVE ACTION ON JUXTAPOSED SLABS

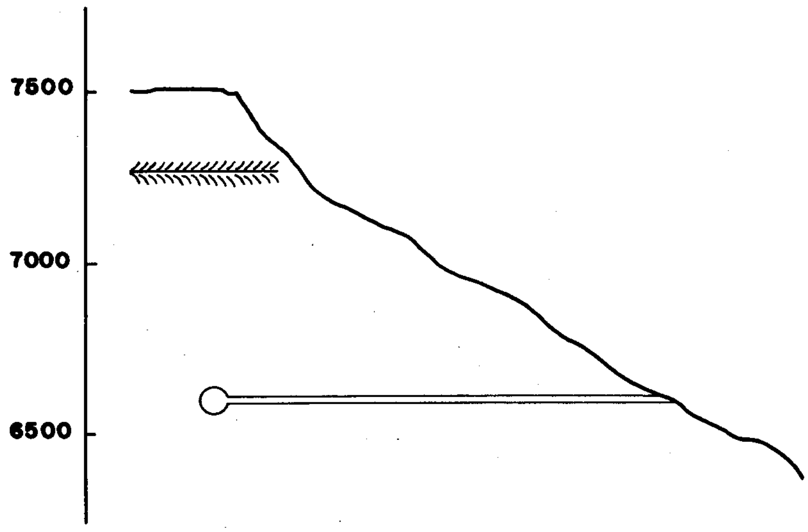


FIG. 12——RANIER SHOT LOCATION SCHEME

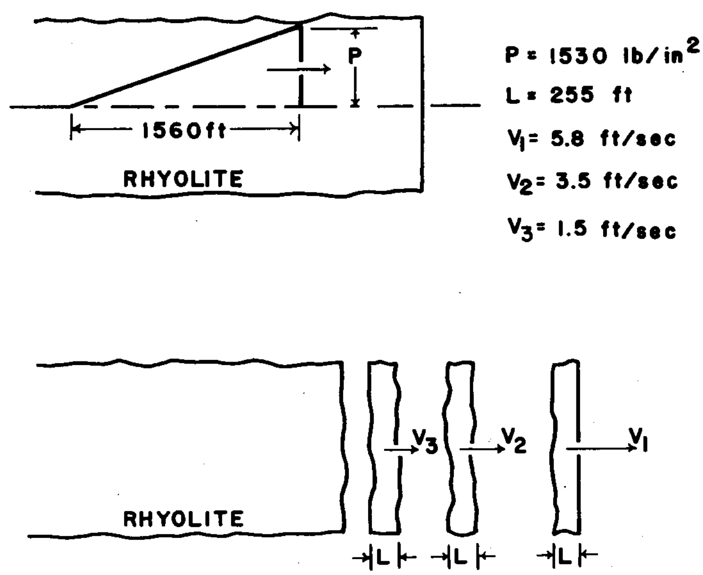


FIG. 13 —— RANIER WAVE APPROACHING FREE SURFACE OF RHYOLITE

DESIGN OF UNDERGROUND OPENINGS FOR PROTECTION

Wilbur I. Duvall
U. S. Bureau of Mines

All underground rock, as a result of nature, is in a state of stress, and any opening created in this rock produces additional stresses in the rock surrounding the opening. Failure of the rock surrounding the opening will occur if the stresses in the rock exceed the ultimate strength of the rock. Thus, the problem of designing safe, stable, underground openings is twofold, (1) the determination of the ultimate strength of the rock and (2) the determination of the maximum stresses in the rock surrounding the opening, regardless of the source of the applied stress, that is, static or dynamic.

First consider the problem of the ultimate strength of rock. Laboratory tests can be performed on small samples of rock to determine such quantities as compressive strength, tensile strength, modulus of rupture, elastic constants, Poisson's ratio, etc. If these quantities are to be of value in the design of underground openings one must assume that these laboratory results can be applied to the in situ rock. Experience has shown that laboratory-determined strength properties predict fairly accurately the in situ strength of the rock, but that laboratory-determined elastic properties do not agree very well with in situ elastic properties. Furthermore, one must assume a criterion of failure for rock. The criterion of failure that is often used is based on the maximum stress theory. That is, rock will fail in tension when the tensile stress exceeds the tensile strength of the rock as determined by a standard flexure test on a sample of the rock. If the tensile stress in the rock is small the rock will fail in shear at a value of compressive stress equal to the compressive strength of the rock as determined by a standard compression test on a sample of the rock.

To show the variability of the properties of rock the data given in Table 1 has been assembled. The strength and elastic properties of rock vary widely from place to place in a given mine and from mine to mine and from rock type to rock type. Because of this fact, good sampling procedures should be used in studying physical properties of mine rock and sufficiently large safety factors should be employed in design work to allow for the spread in the physical property data. The main purpose for this table of data is to show the almost complete overlap of physical property data for the various rock types. Just naming the type of rock does not specify the strength and elastic properties of the rock. These properties can only be determined by testing procedures.

Determining the maximum stress in the rock surrounding an underground opening is a more difficult problem than that of determining the ultimate strength of rock. As yet there has not been developed a completely satisfactory gage and method for measuring directly the absolute static stress in rock. Recent developments in Europe are encouraging, especially the work of the English and Swedish in connection with stress gage developments. Usually one must resort to analytical or model testing methods for estimating rock stresses. To apply these methods requires making assumptions about the rock medium, the opening, and the state of stress prior to mining. Only competent rock should be considered for protection purposes. Competent rock is defined as rock which, because of its physical and geological characteristics, is capable of sustaining openings without

T A B L E 1. - Physical properties of mine rock.

Physical Property	GRANITE Range Median	MARBLE Range Median	SANDSTONE Range Median	LIMESTONE Range Median
Apparent specific gravity	2.6-2.9 2.65	2.7-3.2 2.8	2.0-3.3 2.3	1.4-2.9 2.6
Compressive strength lb/in.^{2}x10^3	20 - 50 30	7 - 34 19	4 - 32 14	1 - 38 19
Tensile strength lb/in.^{2}x10^2	3 - 11 5	8 - 10 8	0.7 - 5 1.5	3 - 6 5
Flexural strength lb/in.^{2}x10^3	1 - 6 3	2 - 3 2	0.4 - 4 1.4	0.3 - 4 2.2
Young's modulus lb/in.^{2}x10^6	1 - 13 4	7 - 12 9	0.8 - 8 2.8	0.4 - 13 8
Longitudinal bar velocity ft/sec x 10^3	6 - 18 11	8 - 18 15	5 - 15 9.5	4 - 19 14

the use of any structural support except that offered by the pillars and walls left in the course of mining.

Any study of the structural stability of underground openings is simplified if competent rock is sub-divided in two groups - massive rock and horizontally-bedded rock. Massive rock is assumed to have relatively uniform physical properties over large areas in all directions, is comparatively free from stratification and, hence, approaches the condition of a perfectly elastic, homogeneous, isotropic medium. Typical examples are massive intrusives such as granite or diorite, massive extrusives such as basalt or rhyolite, some massive metamorphics such as marble and quartzite and some thick-bedded sedimentaries.

Horizontally-bedded rocks include most of the sedimentary rocks and some of the stratified, metamorphic rocks. As the boundary between horizontal strata are planes of weakness, openings formed in bedded rock usually are mined with a relatively smooth, flat roof.

Three types of static stress fields are usually considered when studying the design of underground openings, and these are illustrated in Figure 1. In the following discussion only static stress fields will be considered. The state of stress represented by A would be expected to occur at shallow depths, near vertically free surfaces. The state of stress represented by B would be expected to occur in geologically undisturbed rock over wide ranges of depth. The state of stress represented by C could occur at great depth, in semi-plastic rocks or in geologically disturbed rocks where tectonic forces are still active.

To simplify the problem still further, only openings which are long compared to their cross-sectional dimension will be considered. The cross-sectional shapes of these openings will be represented by circles, ellipses, ovaloids, and rectangles with rounded corners. In addition, if the stress distribution along the length of the opening is assumed to be uniform and independent of the length, the problem of determining stress distributions reduces to one of plane strain and may be solved as a hole in a wide plate subject to a two-directional stress field in the plane of the plate.

Considerable experimental and theoretical work has been done in this field and the results published. Without going into details, some of these results are summarized below.

Consider first single openings in a massive rock. Such an opening can be represented by a hole in a wide plate. The maximum stresses developed around these openings are concentrated near and tangent to the boundary of the opening, are independent of the elastic constants of the medium and independent of the size of the opening. For convenience, stresses in the vicinity of an opening are expressed as a multiple of the average applied stress existing outside the zone of disturbance, commonly termed stress concentration. For an applied stress field of type A the critical tensile stress around openings is usually about 1 and occurs at the center of the roof and floor of the openings. The value of the critical tensile stress is not affected a great deal by the shape of the opening but is affected strongly by the type of stress field. Usually a confining stress greater than $S_v/3$ will eliminate critical tensile stresses for most openings. Figure 2 shows for type A applied stress field, the critical compressive stress concentrations for various-shaped openings as a function of the width-to-height ratio of the opening.

Ovaloids and rectangles with rounded corners are preferred cross-sectional shapes if the width-to-height ratio is greater than 1, and elliptical cross-sectional shapes are preferred if the width-to-height ratio is less than 1.

Figure 3 shows for type B applied stress field, the critical compressive stress concentrations for various type openings as a function of the width-to-height ratio. Again ovaloidal or rectangular cross-sections are preferred if the width-to-height ratio is greater than 1, and elliptical cross sections are preferred if the width-to-height ratio is less than 1.

Figure 4 shows similar data for the case of type C applied stress field. Here the circle or ovaloid is the preferred cross-section shape regardless of width-to-height ratio.

The second problem to consider is the effect of having two or more openings underground which are parallel to each other and separated by a wall of rock. This problem has been studied both theoretically and experimentally. The pertinent results that one obtains from these studies are summarized below.

The stress fields around two or more parallel openings add together to give increased stress concentrations if the thickness of the wall between the openings is less than one diameter of the opening. Furthermore, as the number of openings increases, the maximum stress concentration increases rapidly at first and approaches an upper limit. Figure 5 illustrates these facts for the case of equal-size, equally-spaced circular openings. The conclusion to be drawn from these data is simply that in designing underground openings for protection the spacing of rooms should be such that the pillar width is equal to or greater than the room width.

If this criterion is used then the stress distribution around each room is not affected appreciably by the presence of the other rooms, and critical stress concentrations can be calculated as for single openings.

The third problem to consider is the case of a bedded formation where the bed thicknesses are small compared to room width or height. In this case the boundary between different beds is an inherent plane of weakness and bed separation can and does occur. Therefore one must consider the additional stresses set up in the roof rock as a result of having a gravity-loaded slab of rock over a span equal to the room width and clamped at the edges by the wall rock and the rock above. Experience has shown that the behavior of roof slabs of this type can be predicted fairly accurately by the use of simple beam or plate theory. Figure 6 shows the relation between slab thickness, span length and strength (modulus of rupture) as given by beam theory. From this set of curves one can conclude immediately that unsupported underground openings in thinly-bedded formations are not too satisfactory for protection purposes. If one must place protective underground shelters in bedded formations one should, if possible, select those formations that have bed thickness of several feet or more and which are composed of fairly strong rock.

The fourth problem to consider is the stress distribution that occurs when two or more openings intersect. Very little theoretical or experimental work has been done on this problem. Intersections between openings could easily be sources of high stress concentration and therefore some attempt should be made to design openings so that additional support is obtained at cross-intersections.

How one can approach the problem of designing underground openings for protection has been described in very general terms. Also some of the more important results that have come from stress analysis studies have been pointed out. This paper is concluded with a description of a possible layout of openings that should satisfy some of the requirements for underground shelter. The only consideration given to the possible use of the underground space is that space is required. If the use of the space also dictates the design of the space then these cases would have to be considered individually.

Figure 7 gives a possible plan for floor space layout which satisfies most of the requirements from a stress analysis point of view. The rooms are long compared to their width because less likelihood of failure results if floor area is increased by increasing the length of the room rather than its width. Each room is provided with at least two exits. Each room is separated from the next one by a rib pillar whose width is greater than the room width. Rooms are staggered along main entries to eliminate cross-intersections. The entries to each room are made small to eliminate large unsupported areas at intersections. Only a few rooms are shown, however this pattern could be extended in all directions as desired. The shape of the cross-section of each room could be ovaloidal or rectangular with rounded corners and arched roof. The height of the room is assumed to be between 1 and 2 times the width of the room. Under these conditions the maximum compressive stress concentrations around the rooms probably would not exceed 3.0 for an applied stress field of type B, that is, $S_v = \rho g Z$ and $S_h = S_v / 3$. Maximum tensile stress concentrations for this condition should be less 0.3. If such a design were used at a depth of 1000 ft in massive rock having an average compressive strength of 15,000 psi and average modulus of rupture of 1500 psi the estimated maximum tensile stress around the room would be 300 psi and the maximum compressive stress would be 3000 psi. The estimated safety factors considering static load only would be 5, both for tension and compression. This means that the static working stress on the rock is only 1/5 of the average ultimate strength of the rock. Thus 4/5 of the ultimate strength of the rock is available to withstand dynamic loads or to take care of rock defects.

Some of the important problems which should be studied or solved to aid in developing better methods of designing underground openings for protection are the following:

1. Three-dimensional stress analysis of intersecting openings.
2. Development of a stress gage for measuring static rock stress.
3. Development of a strain relief method of determining absolute static stress in the rock.

DISCUSSION

MR. ROBERT L. LOOFBOUROW: I would like to ask a question, Wilbur, because I think it will be brought out in one of the later sessions. Would you care to comment on the effect of the smoothness of the walls in regard to stability that you obtained?

DR. DUVALL: Well, this is a very big question. All the laboratory work and model studies are based on the idea of a perfectly smooth wall. If you have a circular hole which has been milled in rock, you will have a very smooth surface. Under these conditions, we find both from theory and experiment that our maximum stress concentrations are located tangent to the boundary. If we plot the stress distribution under a load of this type, maximum stress concentration is three and it drops off to one as you move away from the hole.

Some results are coming out of the European countries, where they have stress gauges and strain relief methods for estimating the stress on a rock in an actual underground mine. The stress starts at the boundary, goes up to some point, and then drops off. The explanation for this is that in mining an opening, you do not have a smooth surface but create cracks in the rock. You do not completely break the rock up and it can support some load, but not too much load. It actually does some supporting. Maximum stress is reached at some point, then, farther out around the defects around the opening. I do not know whether we want to get a smoother opening or not. From some of the results coming out of England, it seems to me that a few fractures around the opening are helpful. It keeps the maximum stress concentration down.

MR. WILLIAM BROWN (RAND Corporation): Dr. Duvall, you mentioned earlier that the elastic properties are not the same in situ as you find them in the laboratory. Would you make a comment or two about that.

DR. DUVALL: They are not. We have determined this in two different ways. In a limestone mine where we have a slab of rock over an opening, we have sampled the rock and measured Young's modulus, in the laboratory. Then, we have gone out to the mine and loaded the beam with air pressure above it and flexed it, taking measurements of deflection in order to get a Young's modulus of the slab. These values do not agree. We can take the same samples and break them in the laboratory and get a rupture modulus. We can load the roof slab with air pressure until we break it. They break at approximately the same stress. The strength properties seem to check out here, and the elastic properties don't.

Another case where the elastic properties disagree very badly is in connection with the dynamic work that I have been doing where you actually measure the strain waves from explosions. You take small samples of rock in the laboratory and measure the bar velocity. From the bar velocity, you have to estimate what the free median velocity would be in situ. Then, we actually go out and measure this. It does not agree at all. It can be off by a factor of two very easily. I do not know why. It is very disturbing that we cannot do this, but this is the way life is.

MR. JOHN GILL: You had a statement there about the stress around the opening. I believe you said it went out a distance of the diameter of the opening. Is that right, or is it the radius of the opening?

DR. DUVALL: The diameter of the opening.

MR. GILL: I have two questions. The first question is how is this determined, by actual strain gauges or what?

DR. DUVALL: Well, you can do this theoretically, or you can do it in model studies using photo-elastic models or actual rock specimens. We have done all three. By the time you are out one diameter, you are usually down to where the stress is only 5 to 10 per cent of the applied stress.

MR. GILL: I might suggest that you might be out two diameters if you want to reduce it to zero. You might go down to zero.

DR. DUVALL: Well, I thought this too when I was doing my elastic work. But when I started putting multiple openings in a plate, I found that I got very little increase if they were just one diameter apart. Take, for example, the circle. The stress concentration for a circle is three; and if you go to an infinite number of openings, I think the maximum value you get is 3.27 if the spacing is one diameter. Actually, this is a remarkably small increase when you consider you have gone to an infinite number of openings, which means that the average stress on the pillar is doubled.

MR. EDWIN JOHNSON: What type of modifications will you get in your stress due to plastic flow as a function of time?

DR. DUVALL: What modification will you get if you have plastic flow? Well, I think plastic flow is a lifesaver in mining because we have sharp corners in mining and things like that. We should have very high stress concentrations. Plastic flow must assist us. I do not have proof of that except that we do not get failure where it looks like we should. We must be getting some type of flow in the rock.

CAPTAIN JENSEN (Civil Engineer Corps, United States Navy): Dr. Duvall, being in the Navy, where often the right hand does not know what the left hand is doing, I suspect that the same thing may occur in the Bureau of Mines to perhaps a more limited extent. A few years ago, I was in the organization of the Director of Naval Petroleum Reserves. I went to Rifle, Colorado, several times, where the Bureau of Mines was doing a good deal of work for the Navy in connection with oil shale research under Lloyd Guthrie.

Strain gauges were used a good deal down there. I wondered whether you were familiar with this and whether this information has proved to be of value.

DR. DUVALL: I was familiar with it. I was in on that work. Yes, it has proved to be of value. It did not turn out like we thought it would.

If you are going to simply use strain gauges as a method of strain relief, you have to come up with a better means of mounting the gauge to the rock. This is one problem. You are underground, and you have moisture conditions. If you can dry it out and get the gauge on, then you have to get in around the gauge some way or other to cut the rock. Actually, the way it turned out, we had mounting problems and gauge problems. I still think that some type of strain relief method has a good chance of success. I do not think it is simply a strain gauge on the bare rock.

I mentioned the Hast Gauge and the Potts Gauge. Don't ask me how these gauges work, because I am not sure myself; but what they are doing looks very good to me. They are drilling a hole in the wall or wherever they want to measure their stress. They are putting in some kind of gauge. It may be a crystal type. They put the gauge in, and they put it under a fairly high stress so that it makes contact maybe in three directions. Maybe they are going to measure the change in the size of the hole in three directions, under fairly high stress. Then they will come in with another drill bigger than the original drill and drill out the hole, leaving a cylindrical core of rock. Once this is drilled out, then the inside hole expands. The gauge measures the expansion of this hole. From that expansion, they can compute back to what the original stress was if they know the elastic constant of the rock.

Now, these ideas look good to me. There is a whole field of work here for people to understand how this works and develop the right type of gauge.

I think we in this country should be doing more of this work. We are just getting started on this type of work ourselves at the Bureau of Mines. If we are going underground for protection, I think there could be a whole lot more people working on the fundamental problems of this type rather than just the Bureau of Mines.

MR. T. L. WHITE (Commercial Shearing and Stamping Company, Youngstown, Ohio): I have been designing structures for support of underground openings for a good many years. We have noticed that there are residual stresses in the rock. In the design of structures for underground protection, I wonder if these residual stresses that might exist would be aggravated by the shock waves furnished by an explosion and if that would be dangerous to the structure.

DR. DUVALL: All I can say is that the dynamic load would add to it. I cannot quite see how you could aggravate it. You would just add to it. This is the principle of superposition of stresses. That was what I was pointing out in the static design. From static design, we have certain stresses on a rock. We should know what the stresses are if we are trying to design for protection. We should know how much additional load can be put on top of the load we already have there.

MR. WILLIAM W. PLEASANTS (RCA, Moorestown, New Jersey): Your proposed layout there showed a sharp corner where these tunnels intersect. Yet, you have emphasized a number of times that these are points of stress concentration that should be avoided.

DR. DUVALL: Yes, I didn't take too much time in drawing this. I do not think a sharp corner in the horizontal direction would be as detrimental as a sharp corner around the opening.

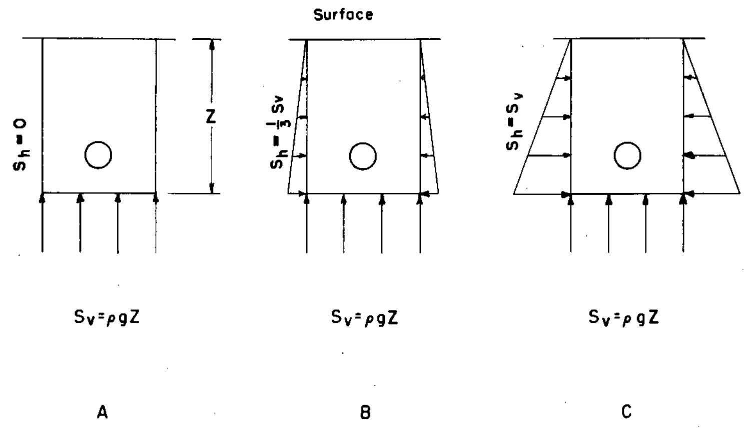


Figure 1. Three assumed types of stress fields.

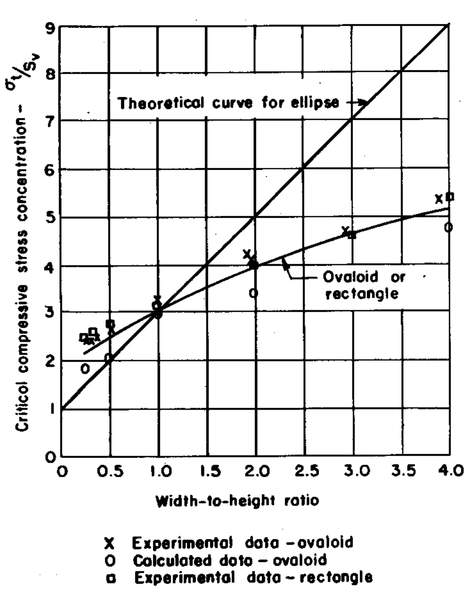


Figure 2 Critical compressive stress concentration for tunnels of various cross-sections-Unidirectional stress field. $S_v = \rho g Z$ $S_h = 0$

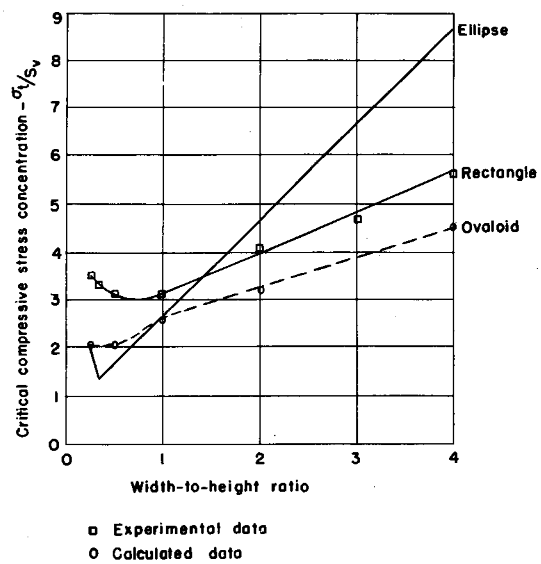


Figure 3 Critical compressive stress concentration for tunnels of various cross-sections — Two-directional stress field. $S_h = \nu_3 S_v$ $S_v = \rho g Z$

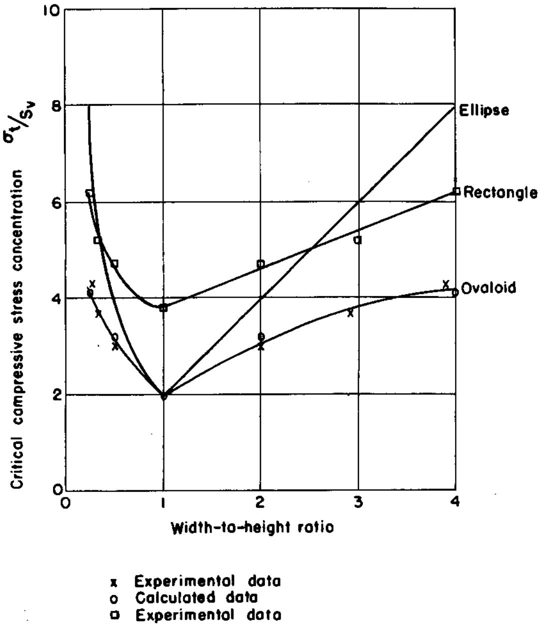


Figure 4 Critical compressive stress concentration for tunnels of various cross-sections-Hydrostatic stress field. $S_h = S_v$ $S_v = \rho g Z$

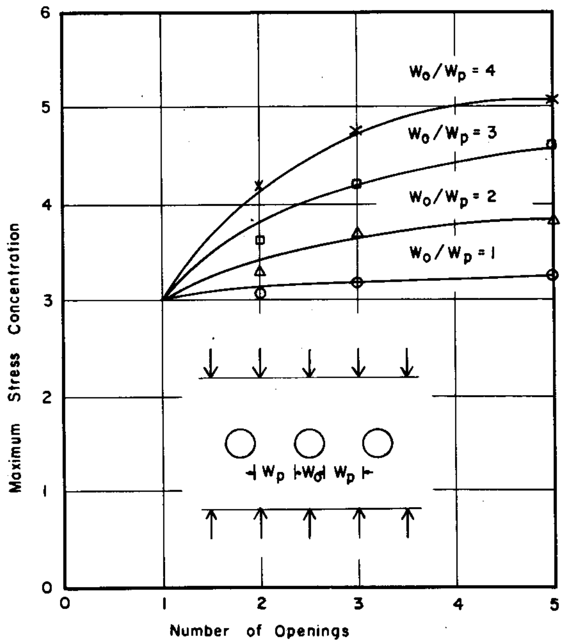


Figure 5 Maximum Stress Concentration as Function of Number of Openings and Opening-to-pillar Width Ratio

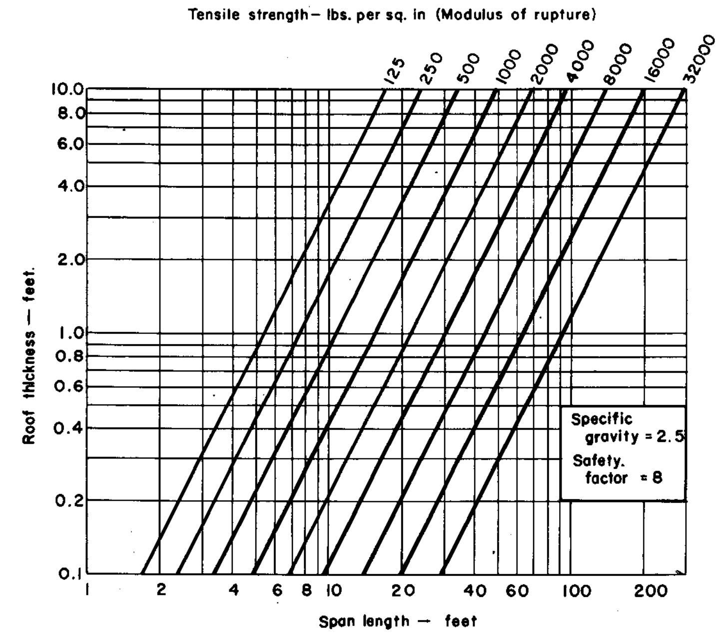


Figure 6 Safe span length for various roof thicknesses and tensile strengths of rock.

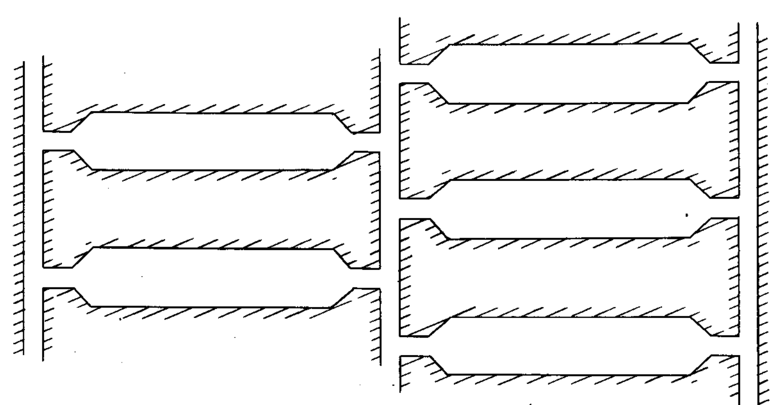


Figure 7 Suggested Plan for Underground Rooms

ROCK STABILIZATION THROUGH BOLTING

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INTRODUCTION

Rock bolting (roof bolting, suspension support, shin plasters, etc.) as a systematic means of ground support has, within the past 10 years, come into widespread use in coal, metal, and nonmetallic mining and tunneling. The extent of the utilization of this recent development as a method of support can be visualized by the statement of the United States Bureau of Mines that in 1958 approximately 40 million roof bolts were used in mining over 218 million tons of bituminous coal.

The concept of fastening loose ground to unbroken and apparently solid ground is probably as old as mining itself; and occasional use has been reported from virtually every mining region in the world. As a means of systematic support over large roof areas, rock bolting waited until the advent of efficient mechanized mining and the development of drills and drilling methods gave it the impetus it needed to become acceptable to the mining industry and to make it economically feasible.

The first published reference (Fig. 1) is an account of tests made in the coal mines of Upper Silesia before the first World War.(1) In the United States during 1920-30, in several underground limestone and lead mines (galena in bedded limestone), developed the practice of bolting rock in localities where roof heights were excessive. Plates holding single bolts of the slotted type, called "shin plasters," were used to fasten loose ground to ground that was obviously secure. The St. Joseph Lead Company, which deserves a major portion of the credit for the pioneer work in bolting, experimented with and adopted structures composed of several bolts having a common channel-section bearing plate (Fig. 2).[2] When bolts were angled, it was found that an arched area above the opening was compacted and stood well without the support of vertical timbers. Roof heights over 30 ft are common in these lead mines, and this new support system resulted in considerable economies which permitted the mining of ores that would otherwise be uneconomical to reach.

In a series of articles in Colliery Engineer[3] in 1945 and 1946, T. S. Beyl, a Dutch mining engineer, proposed a method of supporting roof in the coal mines by the use of vertical bolts having expansion-shell anchors. Beyl's method was based on experiments he made in British mines during World War II.

In 1947, C. C. Conway, chief engineer of the Consolidated Coal Company, adopted a variation[4] of the St. Joseph Lead Company method to a section of the No. 7 mine near Staunton, Illinois. By Conway's variation, instead of using channel-section bearing plates, wooden beams were held against the roof by roof bolts, thus supporting a shale "rash" approximately 36 in. in thickness by securing it to a competent limestone bed. The roof bolts were used in addition to the conventional timbering system used in this mine.

Concurrently with the Conway experiments the Bureau of Mines, United States Department of the Interior, experimented successfully with bolting in combatting the "roof cutter" hazard in the coal mines of northern West Virginia and central Pennsylvania. The Bureau, favorably impressed with the long-established safety record of the St. Joseph Lead Company and recognizing the potential of the roof bolt as a means of roof support, decided to advocate bolting as a means of accident prevention.[5-7] Consequently, twelve field specialists and a research group of four engineers were appointed to assist coal mine operators with roof problems; and in 1950, six additional engineers were added to provide similar services to metal and nonmetallic mines and tunneling operations.

PRINCIPLES INVOLVED

The applications of rock bolting and conventional timbering have a basic difference in concept: Successful rock bolting depends upon being able to make ground self-supporting by consolidation; whereas, with conventional supports it is assumed that the ultimate failure of the ground is inevitable and preparation is made to support it within certain limits after it has ceased to be self-supporting.

In practice, there are two methods of accomplishing reinforcement of rock in place. The first is through the use of a rod having an anchoring device on one end and a rigid bearing device (Fig. 3) on the other end maintaining tension sufficient to compress the intervening ground to the extent that motion between the included laminae or jointing planes is lessened. In this method the rod between the anchoring device and the bearing plate is not in direct contact with the walls of the drill hole unless a partial failure of the intervening ground takes place. Although this method provides ample reinforcement for most ground, it must be borne in mind that its effectiveness is entirely dependent upon the ability to maintain tension between the anchoring device and the bearing plate. This is brought out in Panek's experiments[(8)] with models loaded in a centrifuge demonstrating the necessity of constant compression within the laminae of the beam if the loading is constant. Thus, its permanency is largely dependent upon the efficiency of the fastening device and the creep or plasticity of the rock bolted.

The second method is through the use of continuous fastening devices, or through the use of bolts cemented or grouted their entire length and circumference.

The latter type of bolt is especially useful in situations where the main component of the load applied is normal to the direction of the installed bolt or where, because of creep or plastic flow, it is difficult to maintain constant tension between a fastening device and a bearing plate during the expected life of the opening. Illustrations are the support and stabilization of steeply pitching walls of slopes or shafts, and in horizontally-bedded rocks that have too much plasticity to effect efficient point anchorage.

Another illustration is in driving through the softer rocks, such as shales, where spiling is necessary although it is undesirable to disturb the ground by the action of driving the spiles. Here the method is to drill holes at a flat angle in the skin area in advance of the face and cement tubular bolts into place under low pressure.

The practical mechanics of cementing bolts into the rock has not received as much attention in the United States as it deserves, mainly because such bolting involves a greater cost per bolt and its necessity is sometimes obscured through temporary success with the first method mentioned above. However, this method has been successfully developed in the Scandinavian countries, and two variations were introduced in the United States (Figs.4, 5,6).

The wooden bolt developed by Sterling Lanier, Jr., had the same principle in mind, and it received considerable interest for a time in coal mining because of its low cost. Experience has demonstrated that it is useful under certain unique circumstances in compacting walls and arches, but its tensile strength could not be properly utilized for beam building.

The consolidation and cementation of soft or fractured ground by the Joosten method or variations of this method (infusion of sodium silicate and calcium chloride to form calcium silicate) and by infusion of other resins and glues under pressure, are receiving considerable attention at the present time.[9] It should be pointed out that the application of these methods is dependent upon drilling holes into the ground which is to be strengthened, and that the injection of the solutions or glues can be expedited through the use of bolts, particularly the tubular-type bolts, which also act as a temporary support during the time required for the solution to set or harden.

Relationship to Structural Features

Rock bolts are installed in underground openings on the basis of the relationship of structural features and the physical properties of the rock. Often, detailed geological mapping may be necessary to determine the emplacement of the bolts. Following are some specific cases that have been encountered:

Case 1. - Opening cuts a fault (confined to a single fracture) striking N (at right angles to the opening) and dipping 70°W, which is toward the face. The fault zone is 12 to 36 in. wide, consisting mainly of gouge with occasional angular fragments up to 3 in. in size. The hanging wall and footwall of the fault are solid without associated fractures. Assuming that the main objective is to keep the fragments in the crushed zone from coming out of the roof, the best procedure would be to set lagging in a close pattern across the fault zone and hold them in place by bolts. The bolt in the fault hanging wall could be vertical; the one in the fault footwall would have to be slanted to avoid anchoring in the fault zone.

If the main objective is strengthening the hanging wall and the footwall, the procedure would be to pin the walls together by slanting bolts bearing on the footwall and anchored in the hanging wall of the fault. This procedure may also have the effect of holding the clay and fragments in the crushed zone in place (Fig. 7).

Case 2. - Assuming a fault as in the first case, but the fault zone is 50 ft wide. The fault zone is well crushed and characterized by gouge and angular fragments from 1/4 in. to 3 ft in size. The primary problem here is to obtain good anchorage for the bolts in solid ground in order that they may be used to tie in wire mesh, lagging, or some of the larger angular fragments. Conventional bolts having a limited area fastening device are always dangerous in such ground, and the safest bolt is one of the type that is cemented. Future changes in roof-bolt anchoring devices may result in bolts that will anchor in even the softer material.

Case 3. - In horizontally-bedded rocks where widely spaced joints are encountered, the best procedure is to pin the joints together with the bolt anchored in one stratum hanging wall and bearing on the other. The bolts should be anchored in solid rock above the roof (Fig. 8).

Case 4. - In sedimentary or metamorphic rocks dipping into the face of the opening, the roof may be held by slanted bolts. The bolts should be of sufficient length so that they cross a bedding plane or bedding planes and tie together separate strata or a series of thin layers. The bearing area of each succeeding bolt should be located so that it overlaps a part of the stratum or strata in which the preceding bolt is anchored. The area of overlap is measured from the projection of the point of anchorage in a plane parallel with the bedding. The walls of the opening may be bolted in the same manner.

Case 5. - Assuming a variation of Case 4, whereby the dipping strata are broken by irregular joints. The joints are limited in their continuity and as far as can be seen are characteristic of only certain strata within the opening. The blocks created by the jointing tend to slip out of the roof and walls, and the strata are weak, requiring binding at the bedding planes. Alternating slanting and vertical bolts may often solve this problem. The slanting bolts overlap the anchorage areas of the vertical bolts. All-vertical bolts may be employed in this case if the bolt can pass through one or more bedding planes. Extreme care must be exercised to be certain that all strata where the joints occur are being bolted.

Systematic Support

Over the years the Bureau of Mines and the more responsible coal-mining companies in the United States have found in room-and-pillar mining that the most effective roof support in combatting roof-fall injuries is a support that is installed on uniform centers.

This method is more or less independent of the judgment of the miner, which ordinarily is often influenced by the appearance of the roof, official visits, and the pressure of other work demands. It is a method that provides support by a pre-arranged plan, which is based on knowledge of the roof conditions and on long-term experience and application of the basic principles of support. (The support system in reality becomes an integral part of the mining, such as applied to successful long-wall mining.) Too often in room-and-pillar mining the amount of roof support varies and is primarily the reason for the wide discrepancy between roof-fall accident rates in favor of Europe over the United States. Bolting in sedimentary deposits is an application of uniform-center support that is particularly adaptable to mechanization, because it can be installed at the face fitting into the cycle, thus not impeding the operation of mining equipment.

When using uniform-center support in the average mine, perhaps only 1 of 10 bolts (or props) installed is called upon at a given moment to actually contribute to the support of the roof. If load distribution could be calculated within a reasonable degree of accuracy, it is probable that more efficient bolting patterns could be designed, eliminating many bolts now used; however, heterogeneity of rocks has forestalled such attempts to date. It is known from experience that the more efficient patterns from the accident-prevention standpoint are those wherein the spans between the bolts placed in a given area of exposed roof are approximately equal both laterally and longitudinally.

Bolt Lengths for Beam Building

The length of the conventional bolt is usually determined by the following factors:

1. Available anchorage horizon
2. Available headroom
3. Span of roadway to be supported

It is not always practicable to determine from these factors a common formula for calculating the correct length of the bolt, nor can it be assumed that the bolting will always be 100 per cent efficient, wherein the strength of the compacted beam would vary directly with the square of its thickness. Consideration must also be given to hysteresis of the compressed rock column, slippage of the anchorage, and any significant flexure that occurs in the roof rocks before support is applied.

Of secondary importance is the consistency of the individual rock layers when compacted, provided that the anchoring device is so seated that it will, without slippage, bear a load consistent with the yield strength of the bolt.

Roof bolting in beds where the coal thickness is less than the desired length of bolts presents the problem of using sectional bolts, thereby increasing the cost.

Use of Safety Timbers or Jacks

The Bureau of Mines, as a matter of policy, maintains that a man should never be permitted to work under unsupported roof. Safety jacks, timbers, or posts should be set before any worker advances beyond the last row of bolts. Such devices should be set before bolts are removed and wherever additional bolts out by the face are needed for the protection of the drillers or bolters.

METHODS OF INSTALLATION

The method of installing roof bolts depends on the type of bolts, physical properties of the roof rocks, and the type of drilling equipment available (Fig. 9).

Installation of slotted-type bolts requires some type of compressed-air-operated percussion tool to anchor the bolt by driving the bolt over the wedge within the borehole, such as a hand-held drill or a stoper (Fig. 10). The use of the stoper or other leg-type drill is more favored, owing to the relative inefficiency and ineffectiveness of hand-held drills.

The anchoring and preloading of expansion-type bolts are both accomplished by the tightening process. As the headed bolt is turned, a tapered expansion plug is pulled down into a shell, which, in turn, expands and anchors itself against the side of the hole. This is accomplished by either an air-operated or hydraulic-driven impact wrench or by a portable rotary drill used to drill the bolt holes. The drills are usually mounted on self-propelled equipment and operated either electrically or hydraulically. At the International Nickel Company's mines[10] this operation is done by fitting the stopers with extension handles which act as a ratchet when the drill rotation begins to hesitate.

Vertical vs Angular Bolts

The Bureau of Mines used the system adopted by the St. Joseph Lead Company as a basis for its original specifications and early trials. This system embraced the use of holes drilled approximately at an angle of 60° with the horizontal. As the cost of drilling angular holes is somewhat higher than that for vertical holes, the latter became the more accepted practice. Angular bolting is now used in places where difficulty is encountered in holding the roof with vertical bolts. The practice, though more costly, is particularly recommended at intersections and other points where additional support is needed. The probably valid explanation by stress analysts is that there is a greater likelihood of anchoring such bolts in a compression zone where the rock is least likely to be altered by the mining operations. Dr. Graebach makes a logical case for angling bolts where rock movement can be expected.[11]

Torque Specifications

Experience in using roof bolts in underground mines has shown that to insure good roof control the bolts should be installed or preloaded as soon as possible after the roof is exposed, to minimize the sag of the roof. The amount of this preload is limited by the strength of the bolt or the strength of the anchorage, whichever is less. From a safety standpoint the strength of the bolt should be considered to be not greater than the yield load of the bolt. The strength of the anchorage will vary and therefore should be determined for each mine.

It is common practice to determine the amount of preload on a bolt by use of a commercial-type indicating torque wrench (Fig. 11). This device permits a direct reading of the torque applied to either the nut of a slotted-type bolt or the head of an expansion-type bolt. The reading is taken while the nut or bolt is turning. This reading can be converted to pounds of load by use of a torque-tension relationship which has been determined in underground tests. Underground investigations using 1-in. diameter slotted-type bolts (Fig. 12) showed that the torque-tension relationship was approximately 40 lb of load for each ft/lb of torque applied, provided that reasonable precautions are taken to minimize undue frictional effects. The accuracy of the relationship was ± 27 per cent. For example, a torque of 260 ft/lb applied to the nut will produce a bolt load of 10,000 ± 2,700 lb 90 per cent of the time (Fig. 13). Slotted-type bolts are usually installed at a preload of approximately 10,000 lb, providing a factor of safety of 2, because the yield point of a 1-in. slotted-type bolt is approximately 20,000 lb.

Numerous underground tests using various combinations of 5/8-in. diameter headed expansion-type bolt and shell assemblies showed that the torque-tension relationship for both the regular (SAE 1020-1025) and a high-strength (SAE 1040-1045) bolt was approximately 50 lb of load for each ft/lb of torque applied. The accuracy of these relationships is ± 31 and 34 per cent, respectively (Fig. 14). For example, a torque of 130 ft/lb on a regular bolt will produce a load of 6,150 ± 1,910 lb 90 per cent of the time, and a torque of 175 ft/lb on a high-strength bolt will produce a load of 8,050 ± 2,750 lb 90 per cent of the time. The relationship is valid up to 130 ft/lb for a regular bolt and up to 175 ft/lb for a high-strength bolt. It is not advisable to exceed the above torque values when installing 5/8-in. bolts, because application of torques greater than these will tend to adversely affect the yield point of the bolts. The average yield point of the regular and high-strength bolts tested was found to be 10,770 and 13,880 lb respectively.

All of the aforementioned tests were made with materials in the "as-received" condition. The use of lubricants or hardened washers between the nut and bolt will increase the torque-tension relationships mentioned, and the limiting torques would probably be much lower.[12,13]

Anchorage of Bolts[14-16]

Anchorage with either type of conventional bolt is accomplished through pressure exerted on the side of the hole. The effectiveness of the anchorage depends on the hardness of the rock at the anchorage point, the diameter of the hole in relation to the diameter of the fastening device, the installation procedure, and the efficiency of the bolting materials. The ideal anchorage is one that supports loads up to the yield load of the bolt, with a minimum amount of anchor slip. Anchor slip is defined as the movement of the wedge and of the bolt or the expansion shell with respect to the side of the hole, or movement of the expansion plug within the shell in the case of an expansion-type bolt.

Underground tests have been made to determine the effectiveness of the various methods of anchoring bolts and to determine the factors that vary the effectiveness (Fig. 15).

Tests of 1-in. diameter slotted-type bolts anchored in firm sandstone and shale mine roof showed that (1) anchorages could be developed in either rock that would withstand loads up to the yield load of the bolts; (2) the shape of the slotted end of the bolt affects the ease with which the bolt can be anchored; (3) all bolts for which the driving distance was more than 1.25 in. produced satisfactory anchorages; (4) for the group of bolts that was satisfactorily anchored, the anchor slip was unaffected by the type of bolt but was considerably larger in shale than in sandstone; and (5) the bolt expansion--that is, the amount the wedge expands the prongs in excess of the diameter of the hole--also influences the effectiveness of the anchorage.

Results of anchorage tests by the Bureau of Mines of four makes of 3/4-in. diameter (Fig. 16) headed expansion-type bolts and assemblies show that the anchorage effectiveness of three of the makes of shell was independent of the installed load, the fourth make of shell developed a better anchorage when installed at a load of 8,000 lb, and to obtain maximum anchorage effectiveness a given shell should be installed in the smallest diameter hole that will accommodate it. The results also show that deformation of the expansion plug influences the anchorage. With two makes of plugs a larger displacement was shown for each added increment of load. A relationship between average displacement and increase in bolt load (above the installed load) was established from test results. This relationship can be used to estimate the anchorage effectiveness of bolt-and-shell assemblies anchored in other rock types with different installation procedures.

For comparison with 3/4-in. diameter shells, underground tests were made on various combinations of 5/8-in. diameter headed expansion bolt-and-shell assemblies. Five makes of expansion shells and two types of bolts, regular and high-strength, were employed in the test. The shells were anchored in the same sandstone and shale as the 3/4-in. shells. In the majority of tests, until the bolts yielded, load-displacement curves were similar to those for the 3/4-in. shells. In almost every instance the strength of the anchorage was much greater than that of the regular or high-strength bolts.

Anchorage displacement, as shown by Fig. 17, for a 3/4-in. diameter expansion-type bolt is three times that of a slotted-type bolt anchored in sandstone and twice that when anchored in shale.

Applied Bolting

From the standpoint of utilization, rock bolting can be divided into four distinct classifications, as follows:

1. Suspension. Pinning a loose piece of rock to ground considered self-supporting.

2. Beam Building. Used principally in rectangular openings. The practice of using bolts to cause a series of beds with little or no bond to act as a monolith. Stresses are counteracted by the increased resisting value of the beamlike structure.

3. Reinforcement of the skin area of an opening to provide additional support to counteract the effects of tensile, compressive, and shear stresses.

4. Reinforcement of walls against stresses that are totally or dominantly shear, and against compression.

Suspension Bolting. This type of bolting is the prototype of modern rock bolting and is often called a "shin plaster." It is used with no particular pattern to secure single pieces of loose or questionable material in areas otherwise considered self-supporting. Such bolts should, of course, be strong and have enough anchorage to support the estimated weight of the loose ground with a safety factor of at least 3 to compensate for uncertainties of anchorage and load.

Beam Building. Two aspects of this class of bolting should be considered. The first is that roof bolts may be used in stratified and loosely consolidated rocks to bind them together so that collectively they act as a single beam rather than a series of separate beams. Second, with the bolts that now are considered conventional (slotted and expansion shell) an undesirable disturbance of the strata is brought about by action of the bolt at its anchorage. This occurs because to anchor these bolts properly (so that the anchorage will develop the full strength of the bolt) requires exertion of a tremendous thrust against the rock forming the side of the drill hole. This thrust acts normal to the long axis of the drill hole and tends to create a fissure in the rock at the place of anchorage. This fissure extends parallel to the bedding planes and to the mine roof surface. In pattern bolting, these fissures from the individual bolts connect and form a continuous weak horizon, separating the constructed monolith from the rock above. Therefore, with these bolts it is important that no dependence be placed upon the hanging or "suspension" of a weak shale by drilling a few inches into a massive and strong formation above; the bolts should be long enough so that the total thickness of the resulting monolith renders the beam self-sufficient.

Approximately 80 per cent of the bolting being done in horizontally-bedded mineral deposits is in the category of beam building (Figs. 18 and 19).

Reinforcement of the Skin Area of an Opening. The necessity for this use of the roof bolt comes from two causes: (1) The stresses about mine openings (and in this connection it should be kept in mind that most rocks are very weak and unreliable in tension) and (2) the irregular shape of an opening made by conventional tunnel-driving methods and the fracturing and shattering in the sides of an opening caused by blasting incidental to tunneling.

If an opening is being made in rock that has enough inherent strength to withstand the load of the superimposed strata and it were possible to cut exactly to a semicircular "A" line (Fig. 20), no artificial support would be required. However, where explosives are used this is not possible; the blasting produces a jagged break line exposing and opening the bedding or the jointing planes, and produces fractures that extend into the back and ribs. Therefore, the actual arch line must be considered to be outside of the maximum overbreak or "B" line, and the rock between the "A" and "B" lines may require artificial support. The radius of this "B" line depends upon the effect of the explosives on the bedding or jointing planes. In effectively supporting this type area, a bolt whose penetration is the difference in radius between the "A" line and the deepest penetration of the overbreak multiplied by 2 appears to offer a satisfactory factor of safety and is recommended.[15,16]

When an opening is made, the most intense stress is encountered within a short distance of or at the skin of the opening. The effect of the stress decreases as the rib walls are penetrated; and it is safe to assume that at a distance in the rib, equal to the long cross-sectional dimension of the opening, the stress effect is negligible.

Skin stress can be either tensile or compressive and is variable in that the resultant tension or compression is determined by the degree of arching, the ratio of the vertical load to the horizontal load, and the orientation of the principal axis of the opening to the direction of the acting loads. Of these factors the magnitude of the horizontal load on the opening is always unknown. Consequently, there is never the assurance that the back or roof wall will not be in tension. However, where the roof is in tension it is definite that a relatively short distance from the surface the rock is either under compression or at best a very low tension. Relatively short rods, rarely longer than 6 or 8 ft, will reach rock in which it is comparatively safe to anchor any loose ground that is at the surface of the opening.

This is borne out by the published information on large-bore tunnel driving (Fig. 21) of the New York Board of Water Supply and in the drift and crosscut experience of the Anaconda Copper Company in its Butte, Montana, mines.

When sloughing or spalling occurs about the skin surface of an arched opening, owing either to excessive skin stress or to weathering, bolting has been found to have definite value in compacting and reinforcing this skin area. Although the conventional slotted-type bolt was used successfully for this purpose in the Bureau of Reclamation Duchesne tunnel in Colorado, it is believed that a bolt which fills the entire drilled hole would be more efficient. This appears to be the best application that has been found thus far for the wooden bolt developed by Sterling Lanier, Jr. This bolt is used in the Day mines[17] in the Coeur d'Alene district to stabilize the skin area of arched drift openings in a loosely-consolidated quartzite. It would not be feasible to attempt to anchor a steel bolt in this rock using any fastening devices now on the market; moreover, it should be noted that at least two-thirds of the bolt length bears against the circumferential area of the drill hole.

In coal mines the wooden bolt was used to good advantage to reinforce walls that are weakened because of the incidence of soft clay seams in the coal beds, as in Sakug mine in Austria, iron-ore ribs in Lorain, and to reinforce the walls (ribs) of openings in the room-and-pillar mining of thick coal beds (14 to 30 ft) in Utah.

When the skin surface has deteriorated, but a good anchorage for steel bolts still exists underneath, it is a common practice to use steel screen in lieu of lagging to prevent further progressive sloughing of side walls. At the Lake Shore[18,19] and Macassa mines in the Kirkland Lake gold-mining district of Ontario and in the Sunshine mine of Idaho, steel rock bolts are utilized in combatting the hazard of rock bursts. This is being done from two standpoints:

1. To compact the skin areas of arched openings, attempting to supply enough additional reinforcement so that the opening itself is not the weakest portion in a mass of ground being subjected to extraordinary loads (Fig. 22).

2. Where it appears inevitable that a rock burst will occur during the active life of a drift or crosscut, bolts are used to support a cushioning structure intended to restrain the flying rocks (Fig. 23).

Reinforcing Walls to Resist Shear and Compression. In shafts or steeply pitching stopes there exists the problem of resisting a load resulting from hundreds of feet of an unsupported vertical span. What is needed is a bolt that will better constrain motion in a vertical direction. The conventional bolt will resist shear stresses after the ground starts to move, but only after ground failure when the load can no longer be considered static. Failures of this type have been experienced both in the United States and Canada.

Nevertheless, where the unsupported span is not too high, the effect of strain resulting from the vertical component of this column has been combatted effectively with the ordinary slotted-type bolt. This action was described by V. D. O'Leary[20] in his paper on roof bolting at the Anaconda mines at Butte (Fig. 24):

> Study of the hanging wall seemed to indicate that heavy blocks of ground moved along the joints in a sliding, downward fashion. This movement set up a shearing stress along the contact between the ore and the hanging-wall rock, so that the ore would be dragged down and eventually fall. If the sliding movement in the hanging wall could be temporarily stopped until the fill was placed against it, it seemed reasonable to suppose that stopes could be operated without timber. Rock bolts proved to be the answer to this problem.

The slotted-type bolt is being used successfully in several other ore-mining districts in the United States and Canada to compact the walls of cut-and-fill stopes where the vertical distance between the back and the fill is not excessive.(10)

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DISCUSSION

MR. ROBINSON (Northrop Aircraft): I would like to ask what the typical length would be of these bolts. I saw 4 ft and 6 ft on the slide. What is the longest length you have ever used?

MR. THOMAS: The longest bolt, as far as I know, is about 35 ft; the shortest, in a coal mine, is about 32 to 35 in.

COLONEL ALLEN (North American Air Defense Command): I had some comments relative to the effectiveness of rockbolting for defensive structures. Lest I am putting some wrong information together, I am recalling an earlier presentation that described the wave length of nuclear earthquakes (Raleigh waves, I suppose) of something like 1350 ft , the so-called Ranier shot. That was loaded with rhyolite that had a tensile strength of something like 500 lb /in^2. I am not sure of that.

PROFESSOR RINEHART: That is right.

COLONEL ALLEN: I recall another paper by Dr. Duvall of the Bureau of Mines. In the behavior of these blocks, the spalls with that tensile strength, spalled in pieces that were about 250 ft thick. It does not seem to me that any technique of rockbolting such as we understand it can be at all helpful in maintaining the integrity of underground structures.

Another fallacy occurs to me. That is that the very place that you would anchor these rockbolts are in the areas of higher perturbation and higher disturbance in the plastic and ruptured zones in nearby crater areas. I am only offering these comments, sir, because I am not clear in my mind whether my observations are correct.

MR. THOMAS: I am not sure that they are. At the same time, I know that the roof bolts are being used in several of the existing underground so-called maximum security stations.

PROFESSOR RINEHART: The thing that troubles all of us in these situations is that there are so many qualities of failure. I once tried to become an expert in failure. I discovered that there are so many different qualities of failure that it was difficult to become an expert in that field. As far as static failure is concerned, mine openings and that sort of thing, I think there is no question but that roof bolts have proved themselves.

Now, if you are talking about spalling of large slabs 35 ft to 120 ft thick, you are not going to have roof bolts that long to hold things together. You have to be ingenious about many of these things, you know.

DR. BRODE: Rockbolting is equivalent to placing beams or structural lining around a cavity. The fact that wave lengths are thousands of feet long and spalls are hundreds of feet thick (if such can be called spalls) for nuclear burst ground shocks is very encouraging from the standpoint of maintaining the integrity of an underground structure. Like Noah's Ark on the ocean--if the waves are long compared to the size of the Ark the waves may have huge amplitudes and still be no threat to the hull or its occupants, but if the waves are of wave length comparable to the dimensions of the craft, besides shipping a lot of water, the Ark may actually break up. In the same way, an underground structure will be most vulnerable to wave lengths comparable to the dimensions of the structure. For high frequency components of the wave, therefore, for wave length elements shorter than the gross dimensions, considerable spalling could be expected, but against this mode of failure rock bolting should be very effective.

Since bolting down the whole overburden is impractical, there is slight danger of exposing the buried ends of rock bolts to strains significantly different from the rest of the bolted region, therefore, bolting in much greater depth than the span of the cavity appears to be neither necessary nor particularly practical.

FIGURES

(All Figures courtesy Bureau of Mines, U.S. Department of the Interior, unless otherwise noted.)

16. Relationship for estimating anchorage effectiveness of 3/4-inch bolts.

17. Comparison of anchorage test results for 1-inch slotted-type bolts and 3/4-inch expansion-type headed bolts.

18. Mechanical loading pattern bolting using 6-foot-long vertical slotted-type bolts (laterally and longitudinally with respect to the direction of roadways) to support a thinly bedded shale roof. Tennessee Coal and Iron Division, U. S. Steel Company.

19. Pattern bolting with 1-inch 8-foot-long slotted-type bolts with link mesh screen to control light sloughing. Howe Sound Company.

20. Bolting applied to tunneling.

21. Tunnel roof support. No. 5 mine ties and angle washers are shown. View is upward along line of tunnel.

22. Use of rock bolts to compact portions of the skin area in anticipation of extraordinary loads.

23. Use of rock bolts to support a cushioning structure to contain flying rock so as to prevent injury to personnel. Lake Shore Mines Limited.

24. Bolting ore back to hanging wall. The Anaconda Company, Mining Department.

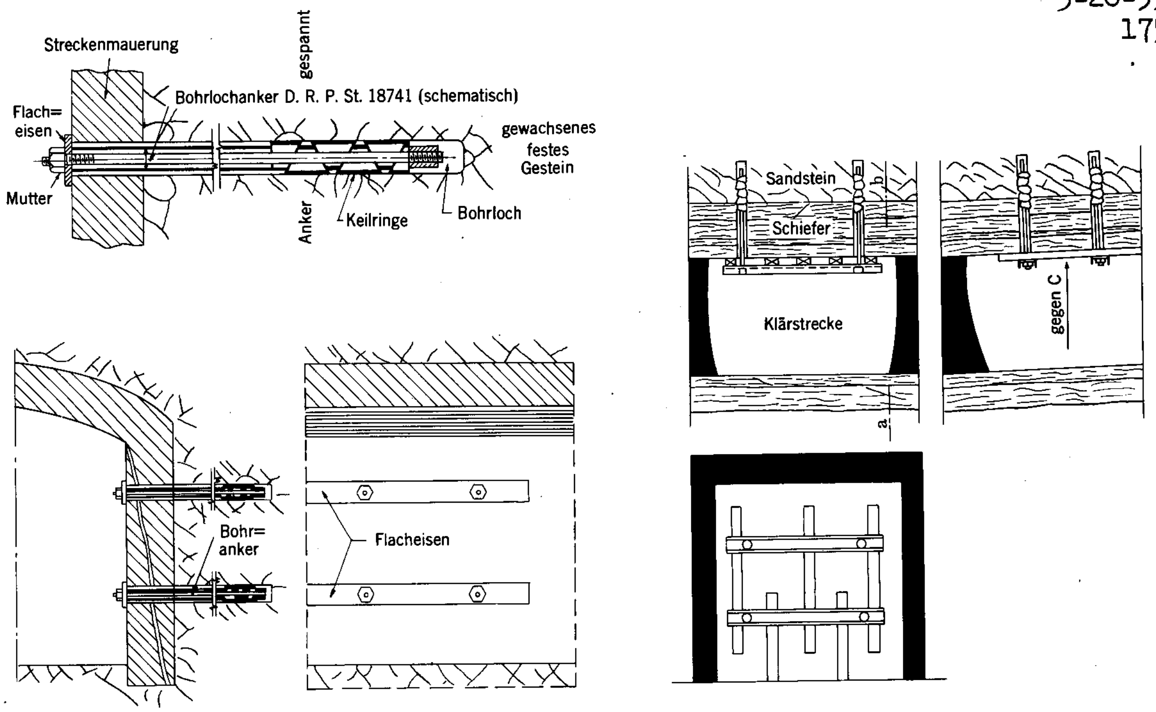
Streckenmauerung
gespannt
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Fig. 1

Fig. 2

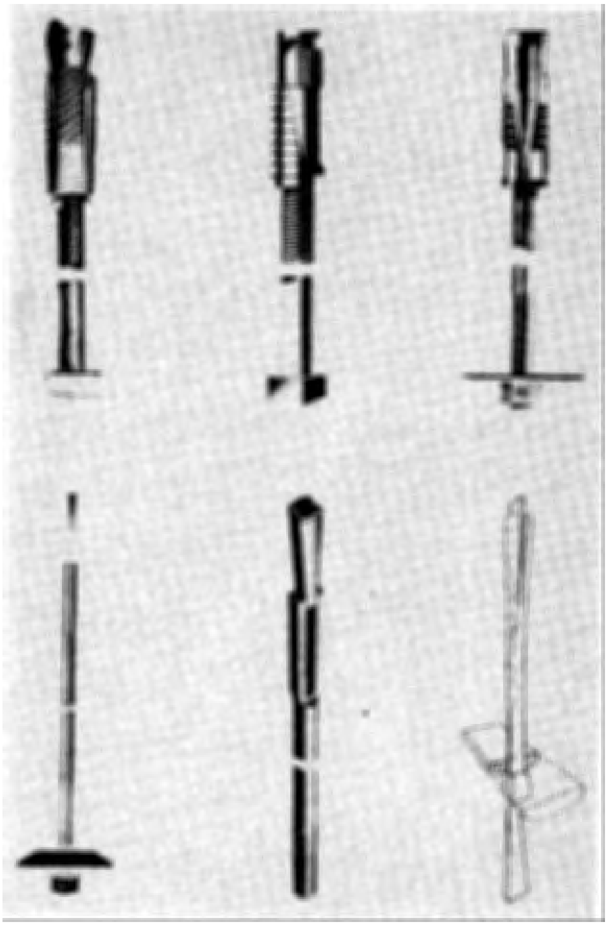


Fig. 3

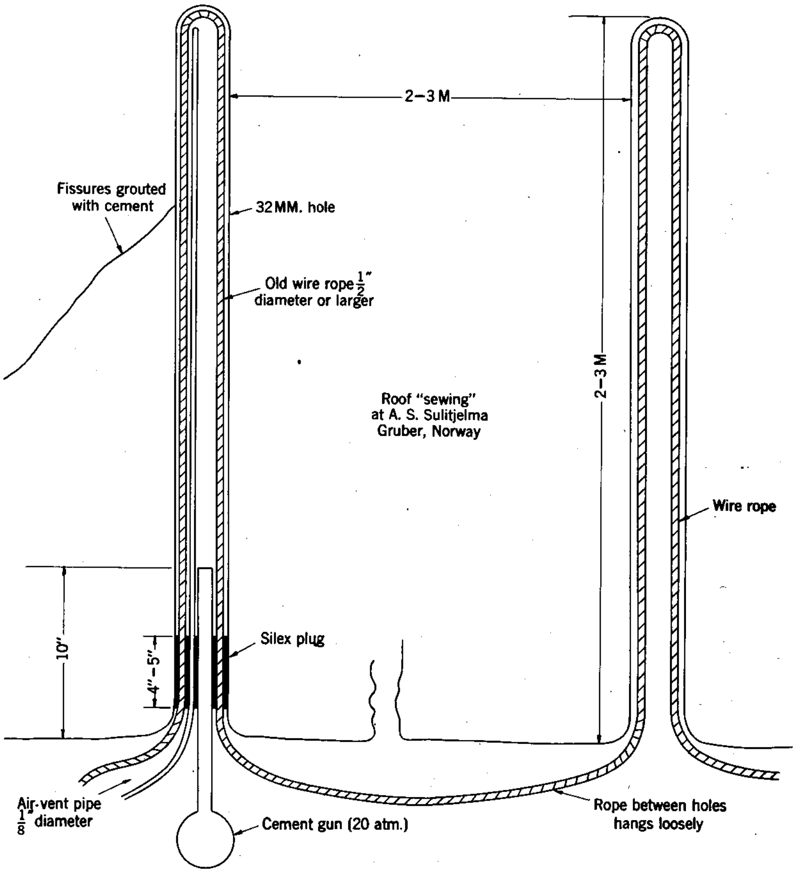

2–3 M
Fissures grouted with cement
32 MM. hole
Old wire rope 1/2" diameter or larger
Roof "sewing" at A. S. Sulitjelma Gruber, Norway
2–3 M
Wire rope
10"
4"–5"
Silex plug
Air-vent pipe 1/8" diameter
Cement gun (20 atm.)
Rope between holes hangs loosely

Fig. 4

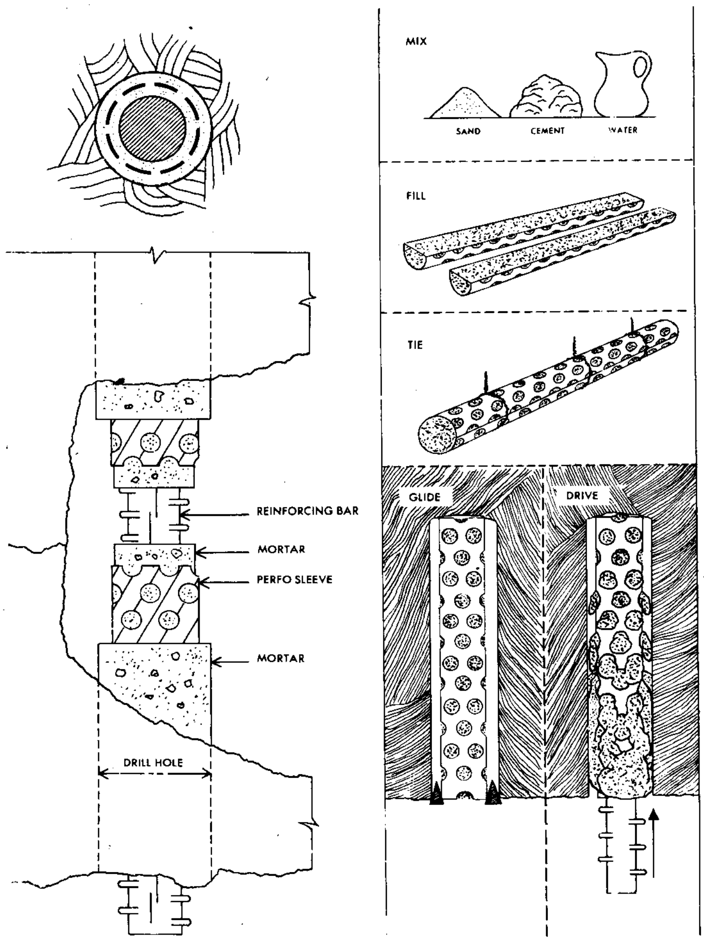

MIX
SAND
CEMENT
WATER
FILL
TIE
GLIDE
DRIVE
REINFORCING BAR
MORTAR
PERFO SLEEVE
MORTAR
DRILL HOLE

Fig. 5

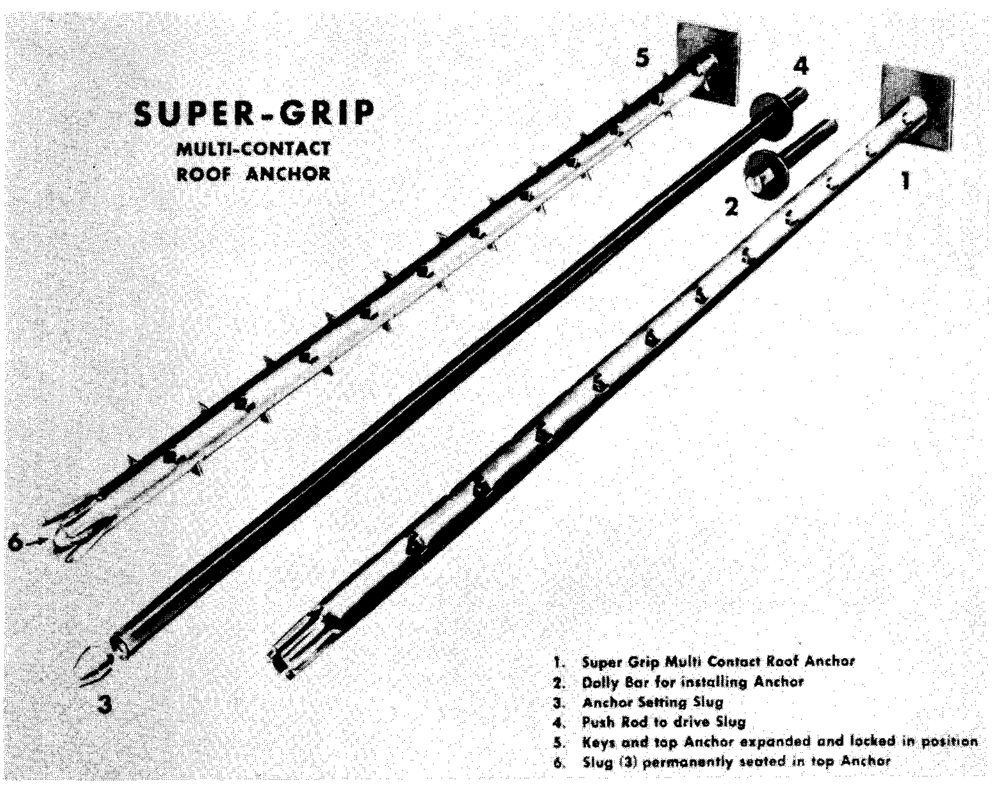

SUPER-GRIP
MULTI-CONTACT
ROOF ANCHOR
1. Super Grip Multi Contact Roof Anchor
2. Dolly Bar for installing Anchor
3. Anchor Setting Slug
4. Push Rod to drive Slug
5. Keys and top Anchor expanded and locked in position
6. Slug (3) permanently seated in top Anchor

Fig. 6

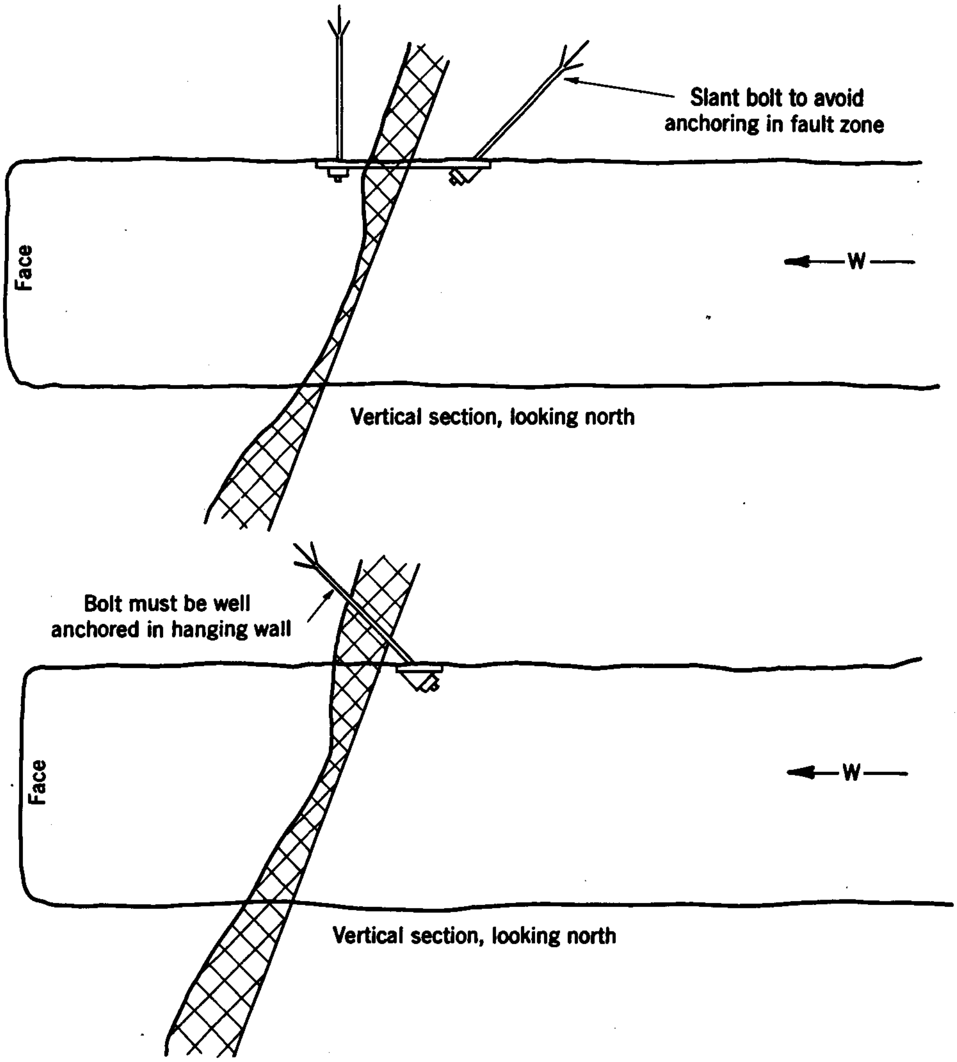

Slant bolt to avoid anchoring in fault zone
Face
W
Vertical section, looking north
Bolt must be well anchored in hanging wall
Face
W
Vertical section, looking north

Fig. 7

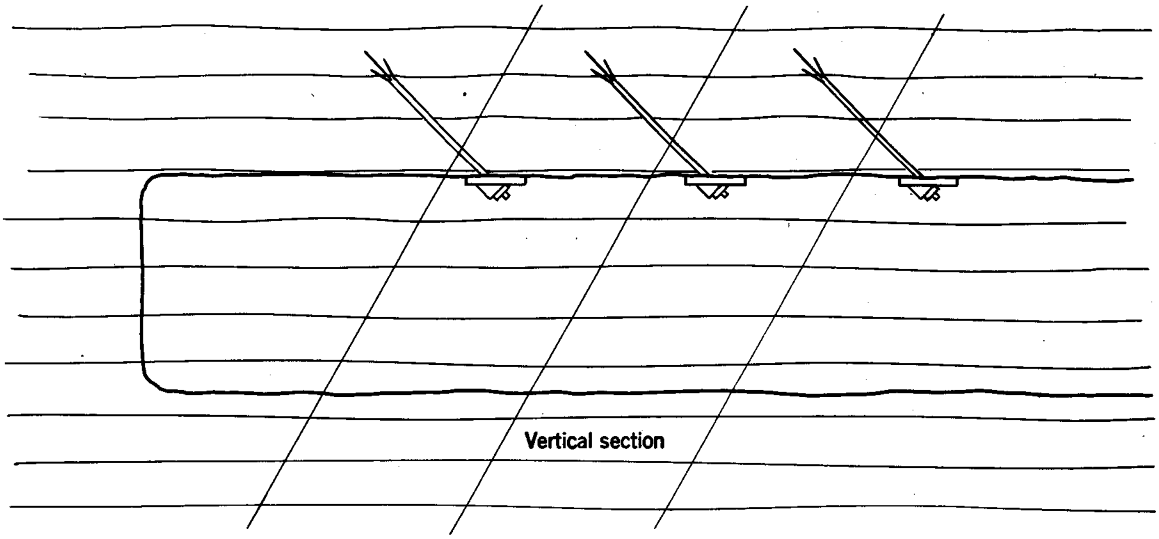

Vertical section

Fig. 8

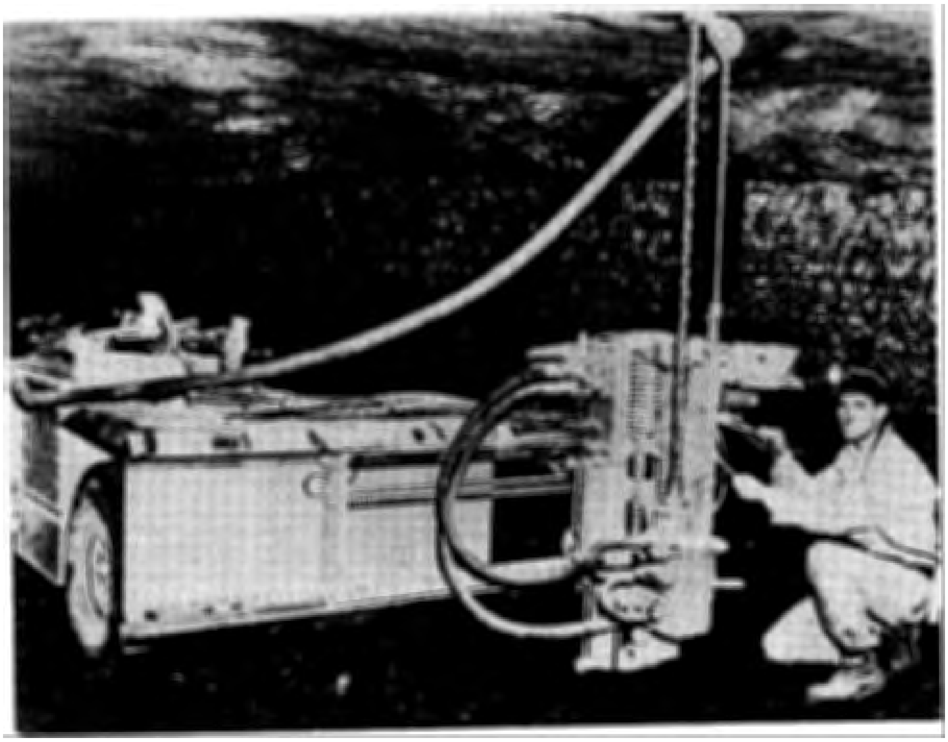


Fig. 9

Fig. 10

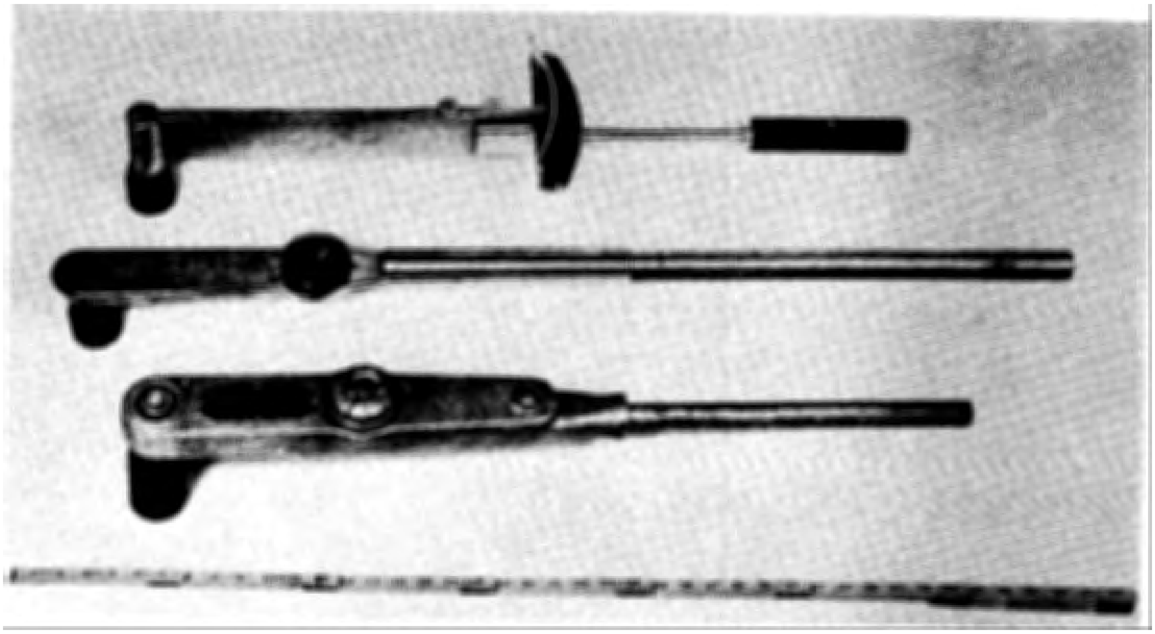


Fig. 11

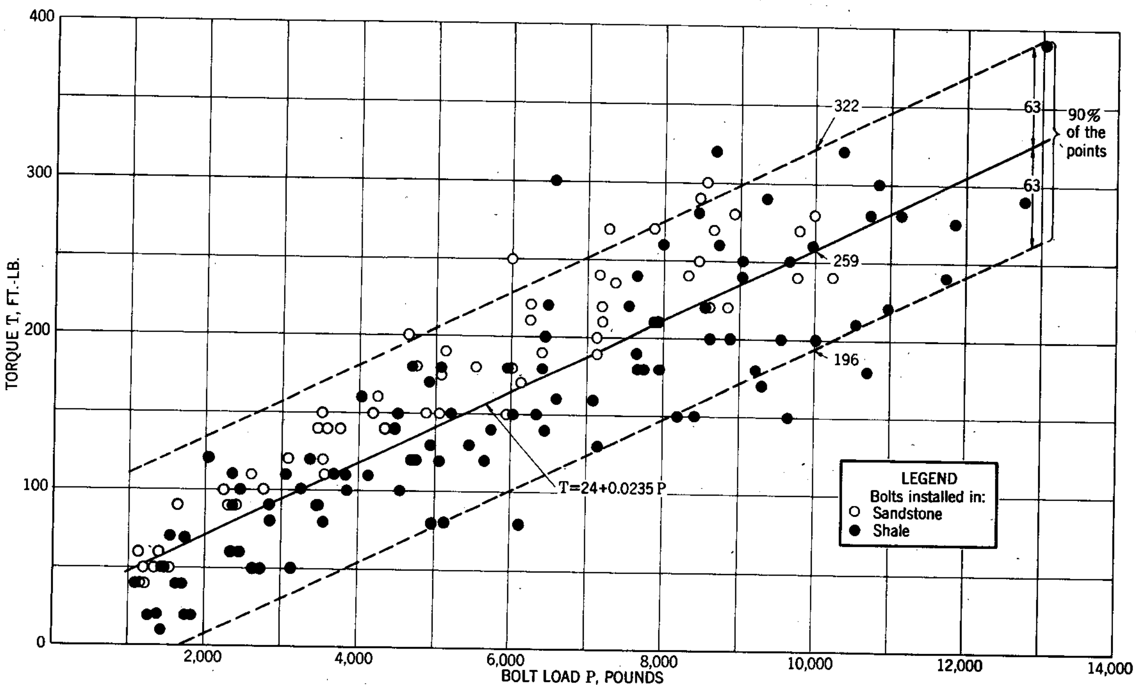

TORQUE T, FT.-LB.
BOLT LOAD P, POUNDS
0
100
200
300
400
2,000
4,000
6,000
8,000
10,000
12,000
14,000
322
259
196
63
63
90% of the points
T=24+0.0235 P
LEGEND
Bolts installed in:
Sandstone
Shale

Fig. 12

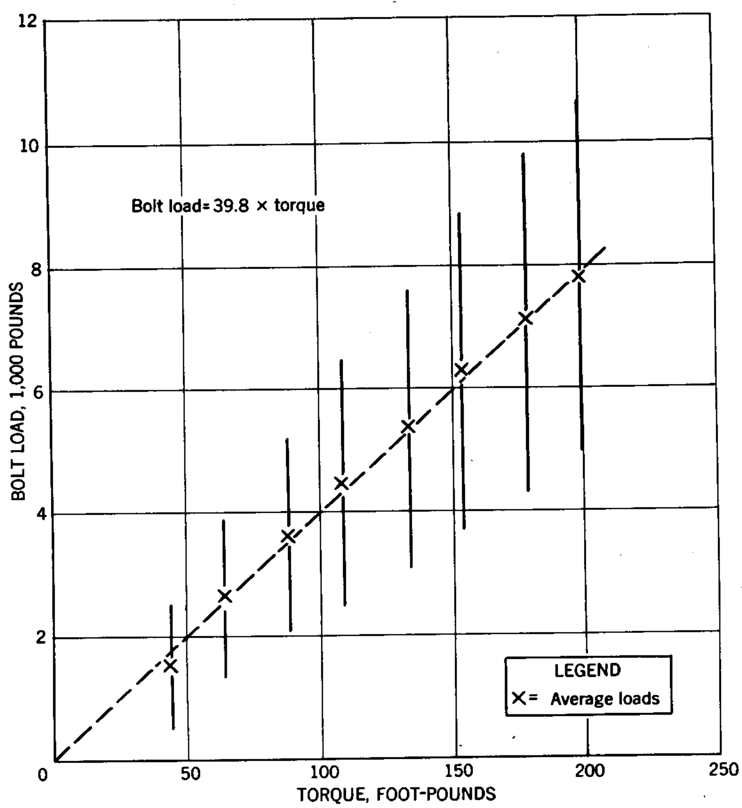

Bolt load= 39.8 × torque
BOLT LOAD, 1,000 POUNDS
TORQUE, FOOT-POUNDS
LEGEND
X= Average loads
0
2
4
6
8
10
12
50
100
150
200
250

Fig. 13

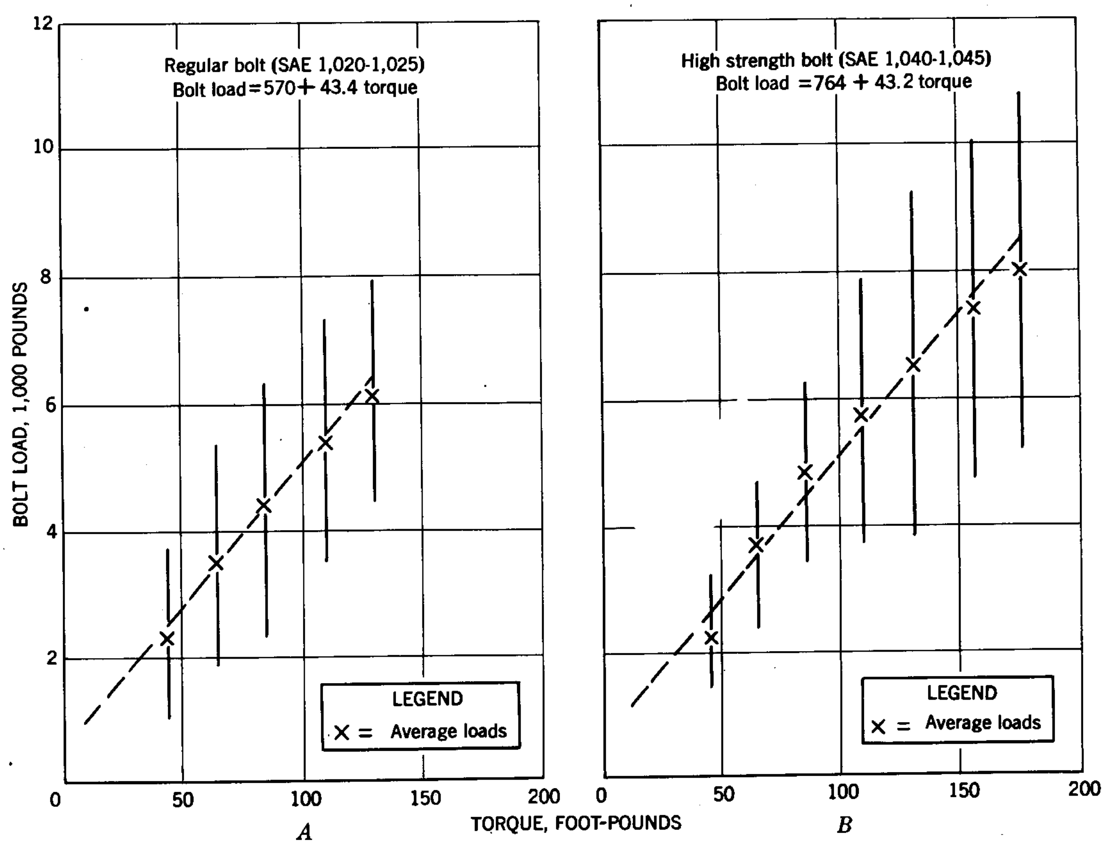

Regular bolt (SAE 1,020-1,025)
Bolt load = 570 + 43.4 torque
High strength bolt (SAE 1,040-1,045)
Bolt load = 764 + 43.2 torque
BOLT LOAD, 1,000 POUNDS
TORQUE, FOOT-POUNDS
LEGEND
X = Average loads
LEGEND
X = Average loads
A
B

Fig. 14

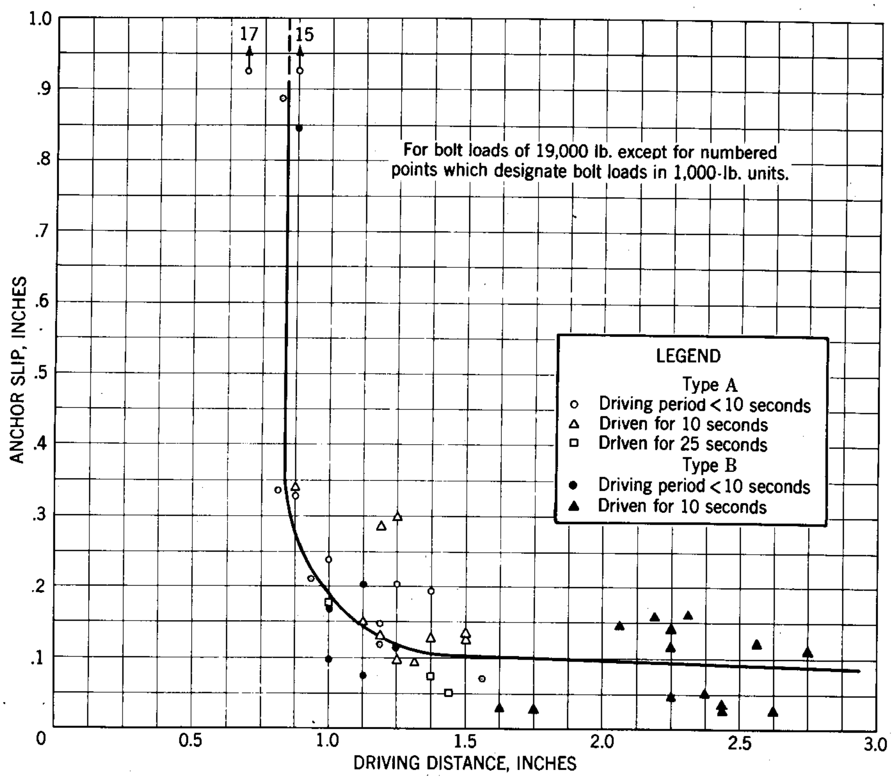

1.0
.9
.8
.7
.6
.5
.4
.3
.2
.1
0
17
15
ANCHOR SLIP, INCHES
For bolt loads of 19,000 lb. except for numbered points which designate bolt loads in 1,000-lb. units.
LEGEND
Type A
Driving period < 10 seconds
Driven for 10 seconds
Driven for 25 seconds
Type B
Driving period < 10 seconds
Driven for 10 seconds
0.5
1.0
1.5
2.0
2.5
3.0
DRIVING DISTANCE, INCHES

Fig. 15

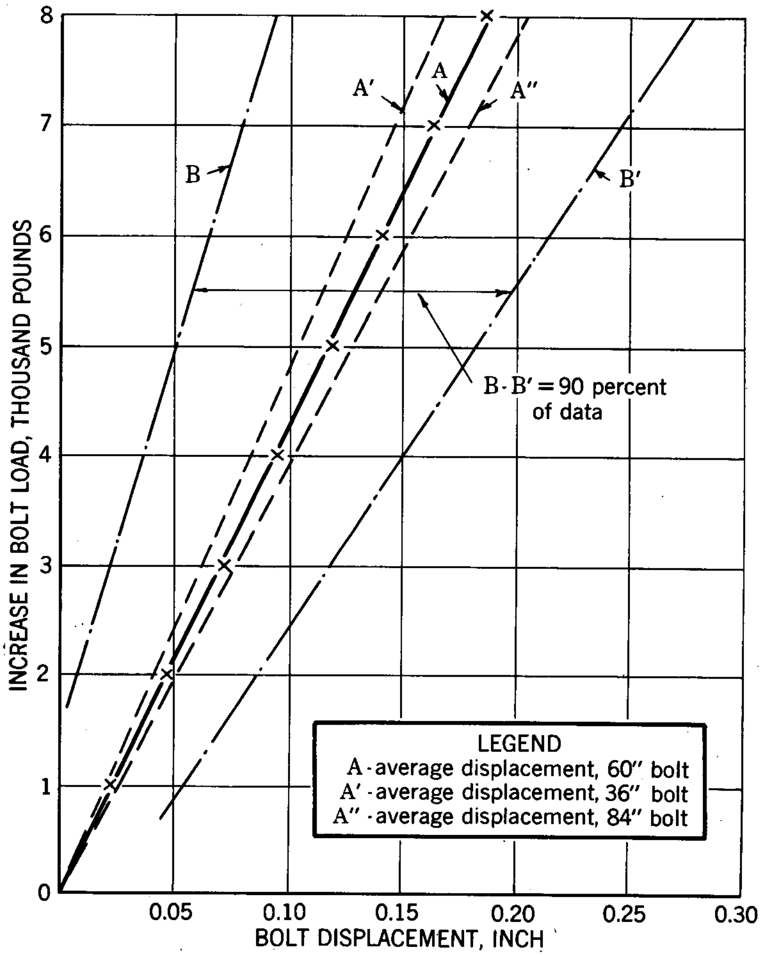

8
7
6
5
4
3
2
1
0
INCREASE IN BOLT LOAD, THOUSAND POUNDS
A'
A
A"
B
B'
B-B' = 90 percent of data
LEGEND
A - average displacement, 60" bolt
A' - average displacement, 36" bolt
A" - average displacement, 84" bolt
0.05
0.10
0.15
0.20
0.25
0.30
BOLT DISPLACEMENT, INCH

Fig. 16

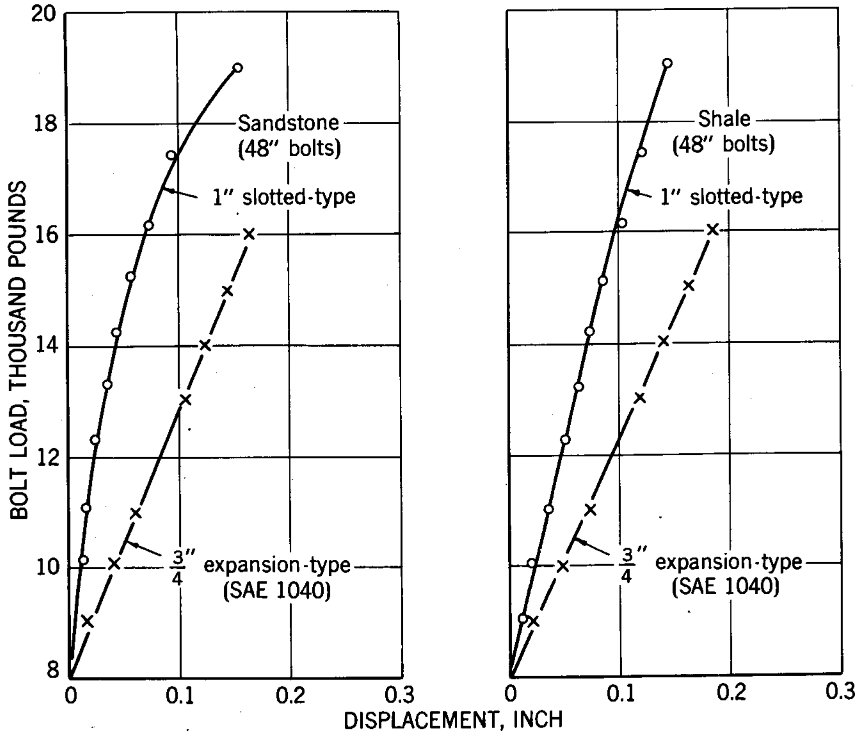

BOLT LOAD, THOUSAND POUNDS
DISPLACEMENT, INCH
Sandstone
(48" bolts)
1" slotted-type
3/4" expansion-type
(SAE 1040)
Shale
(48" bolts)
1" slotted-type
3/4" expansion-type
(SAE 1040)

Fig. 17

Fig. 18

Fig. 19

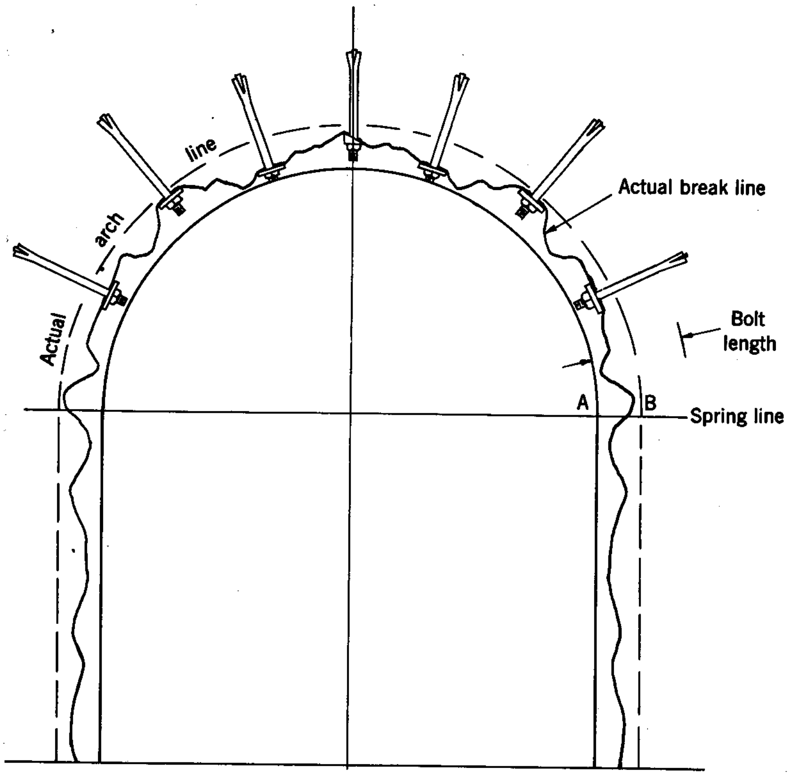

Actual arch line
Actual break line
Bolt length
A
B
Spring line

Fig. 20

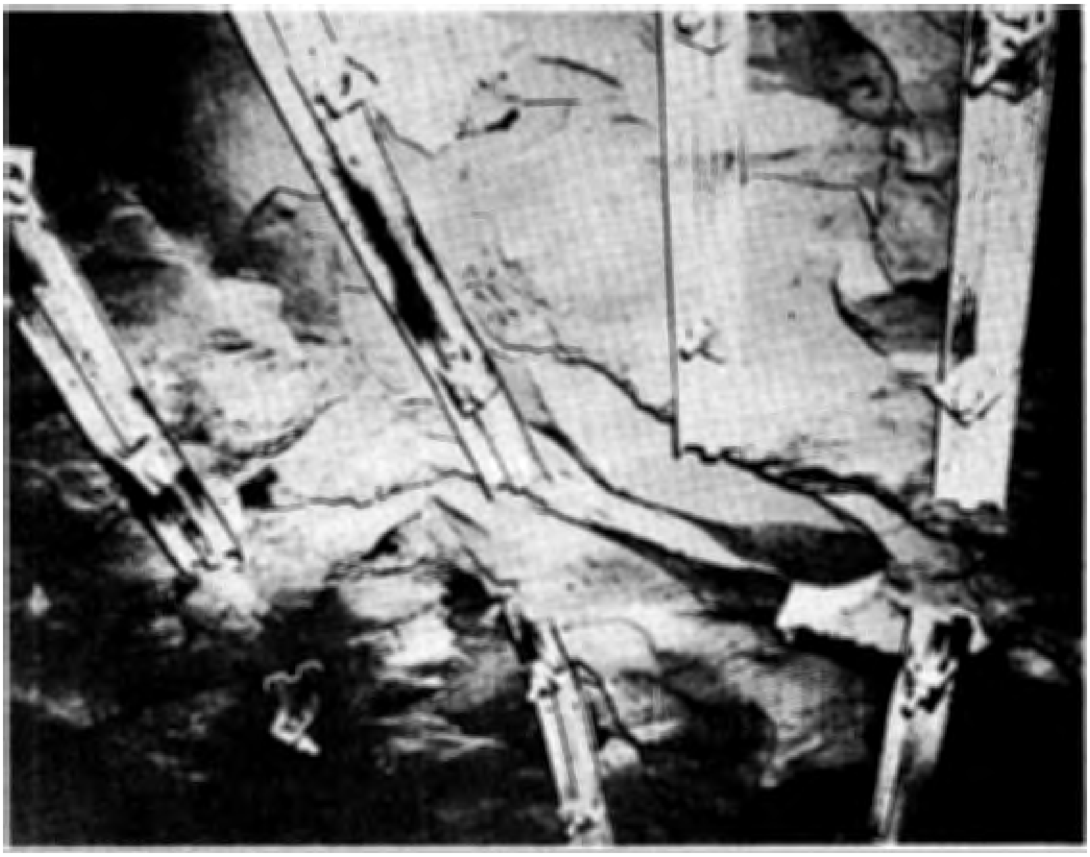


Fig. 21

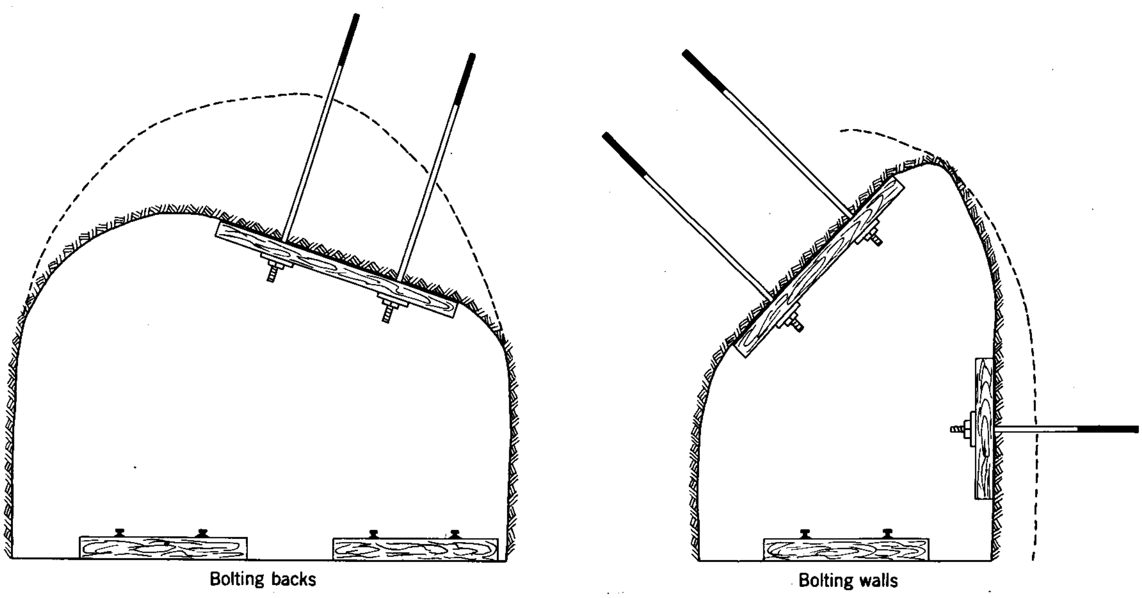
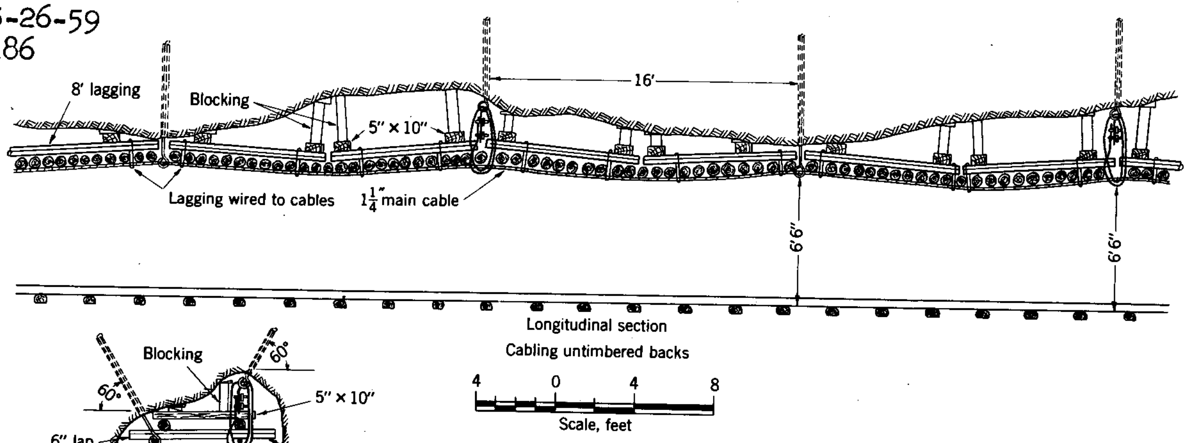
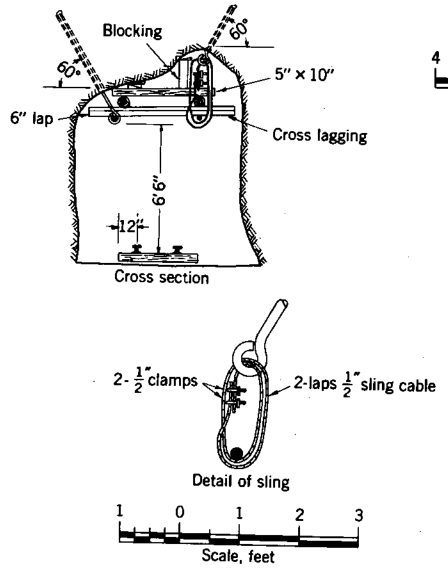


Fig. 22





Method of installation

1. Use 7' Foran pins.
2. Holes drilled 60° at right angles to drift at 16' intervals.
3. All pins put in ground full length, using slings to correct for height.
4. Slings are ½" cable lapped twice, using two ½" clamps.
5. Cross-lagging blocked down with 8' lagging, wired to cable and blocked to back.
6. Anchor pins and system of anchoring (see Dwg. No. 9A15).
7. Cables 6'6" above ties.
8. Drift sets with 8" caps to be placed under cables at about 30' intervals as extra support.

Fig. 23

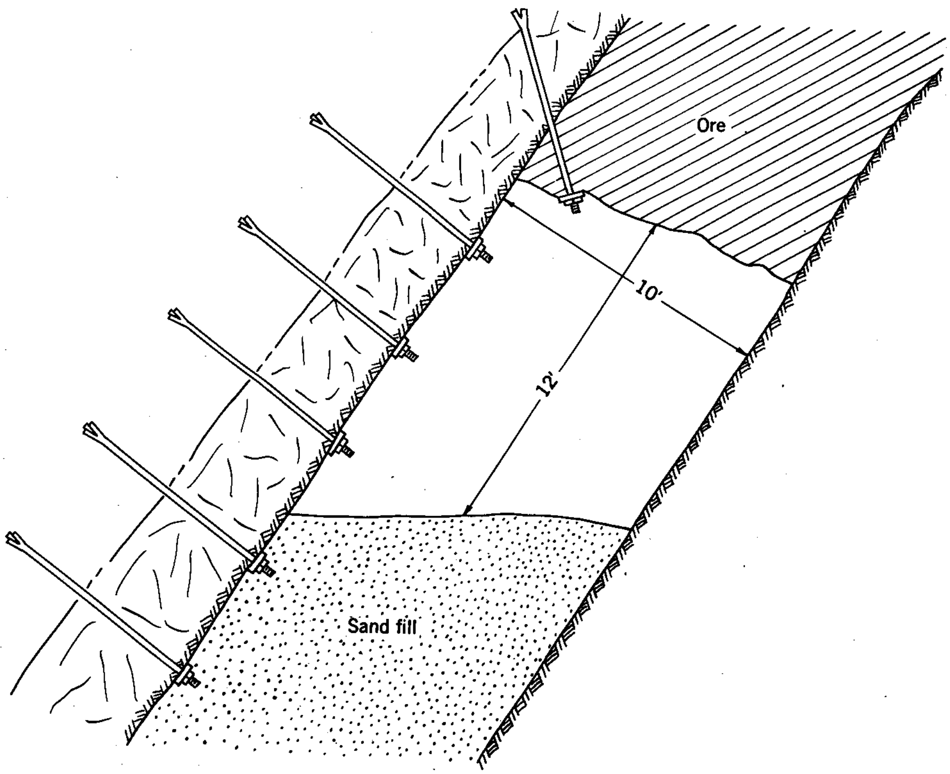


Fig. 24

PROBLEMS OF INSTRUMENTATION IN THE UNDERGROUND AREA

L. M. Swift
Stanford Research Institute
Menlo Park, California

Before discussing the problems of instrumentation in the underground area, we should examine for a moment the purpose of such instrumentation. The experimenter is tempted to consider the data he produces as the end product, but this is far from the case. In a medium such as the soils involved in protective construction, it is patently impossible ever to amass enough data by direct experiment to cover any reasonable fraction of the cases which will be encountered. The experimental data must be used as a base for, and/or as a check on, predictions derived from analytical procedures. But the analyst is faced with a staggering number of possible parameters, so that, even if he knew them all, even the modern computer would be overloaded with some of the simpler problems of wave propagation in the earth at high stress levels. At the best, then, he must compromise by ignoring some of the characteristics of the medium which he knows do, in fact, exist, and hope to approximate the real case with his simplified solution. It is here that the experimental data serve both as a starting point and as a check upon the ultimate results. In the meantime they serve the designer, who must go ahead and design structures this year, as typical sets of inputs, from which he takes a flying leap to a prediction of inputs in the case of interest.

The experimenter must not forget, however, that he does not know to what use his data may eventually be put. Data taken years ago are repeatedly being reworked from new viewpoints--if those data were not preserved in their "raw" form, much of their value may be lost.

Underground instrumentation used in weapons effects tests is largely for the specific purpose of describing the transient effects observed in the free field or on or in underground structures as a result of a large explosion, usually nuclear. There is another branch, that of the measurement of the characteristics of the medium, which we may consider briefly later.

The present techniques used in the primary application are predominantly those of remote recording, as a function of time, of the phenomena detected by gages at the points of interest. The systems used in transmission and recording of the data are, on the whole, satisfactory and well proven. They involve almost universally the use of some type of wire transmission of carrier-frequency signals modulated by the basic transducers, with direct recording on recording oscillographs and/or wide-band frequency-modulated recording on magnetic tape, for subsequent play-back for examination and data analysis. Typical installations will involve from 40 to 150 channels of data recording in a single recording shelter. There has been practically no use of signal-frequency transmission or of radio telemetering of signals in this field, nor is it considered likely that such will be used widely. The major problems in transmission and recording are the protection of the equipment from blast, shock, and radiation, and the protection of electronic circuits from the transient electrical signals experienced at zero time. These problems have been solved satisfactorily for most applications by obvious means, but must be considered in the planning of each experiment.

In general, we feel that newly-developed sensing devices should be compatible with existing proven transmission and recording systems wherever possible.

There has been some use of passive (recoverable) self-recording gages of several types, particularly in the field of structural response, but their use is limited by the problems of recovery.

In the department of basic transducers, the favorite is and has been some form of balanced variable reluctance transducer. This includes three-terminal half-bridges and variable differential transformers and the like. Strain-wire type variable resistance bridges such as SR-4 strain gages and bonded or unbonded strain wire transducers of other types have been used successfully. However, they must be used with caution, because of their lower sensitivity and consequent low signal-to-noise ratio, and because of their greater sensitivity to damage by electrical transients. Variable resistance devices based on potentiometers have been found applicable where the motions sensed and the forces available are adequate for their actuation. There has been practically no use of variable capacitance transducers--they usually require a very high carrier frequency to keep their impedance moderate, and this complicates transmission over long lines.

Whatever the basic principle of the transducer, it serves to modulate a carrier signal which is generated at the recording point, or locally generated, and which is transmitted to the recording point over three- or four-wire shielded cable. This cable is usually buried to protect it from blast and thermal effects, and from pre-shot traffic. The latter constitutes a real hazard, even on the remote islands of the Pacific Proving Grounds.

Turning now to the meat of the problem, the phenomena of interest in the underground free field may be classed as (1) input loading, (2) soil stress, (3) soil strain, and (4) soil particle motion. These are all obviously interrelated, but their relationships are not simple. At the levels of interest, the earth is far from a linear, elastic medium, and is never homogeneous. The variations of these phenmena along the travel path (with depth, for instance) depart markedly from those predicted by elastic theory. Fortunately, this departure is usually in the direction of greater attenuation of peak values and a general degradation of wave form, but complex changes are introduced by refraction and reflection from changes in soil character. The input loading may be considered as being air pressure in all cases, but with surface or underground shots part of it may be at such a level that its measurement is impossible--any physical gage will be destroyed. Outside the crater, air pressure gages of adequate range and response time are commercially available, and measurements of air blast have been made successfully up to the very edge of the crater of a large surface shot.

As the loading penetrates the medium it has the form of a stress wave, initially identical to the input pressure, but rapidly modified by geometry and by dissipation. Stress measurements are the "stickiest" of all. Since stress is a tensor rather than a vector, full description of the stress at a point requires five or more independent measurements. With real gages, this requires that they be closely enough grouped to be considered as being at one point, but far enough separated that they do not interfere with one another! Granting this as a possibility, or assuming that the measurement of one stress tensor is adequate, the gage must be matched to the medium and must be installed so that it does not disturb the characteristics of the medium. This is a large order! One form of stress gage has been used rather extensively, nevertheless. It was originated by Professor Carlson, of the University of California, and consists of two rigid circular plates joined at their periphery by a flexible seal, forming a "sandwich" with a thin layer of fluid between the plates, with a pressure transducer sensing the fluid pressure. When ideally installed, the fluid pressure is a measure of the stress tensor along the axis, but serious difficulties have been experienced with the use of this gage in the free field, difficulties shared with any substitutes therefor which have been suggested to date.

Another type of measurement has been used in lieu of a stress gage. This consists of measuring the transient pressure at the bottom of a fluid-filled hole, or in a fluid-filled sealed cavity in the earth. In general, the interpretation of the data obtained has not been very satisfactory.

The stress at any point in the soil results in a strain, not, however, proportional to stress since the soil is usually non-linear and/or subject to viscous or plastic flow. Strain gages using spans ranging from a few inches to many feet have been used with reasonable success by several agencies. The longer ones measure integrated strain, or relative displacement. The shorter ones are usually taken to represent the strain at their mid-point. These, however, suffer from the disadvantage that if they are installed in a small hole they measure the strain in the backfill, which may be different from that of the adjacent formation. To attach them to the undisturbed soil, it is necessary for a man to install them, calling for a large hole which represents a large disturbance in the medium, even after backfilling.

The stress wave passing a point results in a transient motion of each soil particle. This motion can be described in terms of particle acceleration, velocity, or displacement as a function of time. Mathematically, these are rigorously related. The preponderant majority of experimental measurements of motion on weapons effects tests have been of acceleration. Reliable accelerometers are commercially available, and they appear to suffer little from installation problems. When efforts are made to derive particle velocity and displacement from acceleration data, however, errors of integration become important. This is largely because a typical record of acceleration as a function of time shows a large first pulse of short duration followed by small, lower frequency events which are equally important, especially to terminal velocity or peak displacement. Superb resolution and dynamic range in the acceleration measurement are required, therefore, for useful determination of velocity and displacement by integration.

Commercially available velocity gages such as moving-coil seismometers are too limited in frequency response and dynamic range for this use, and other types are complex and expensive. A new model is under development, but field tests to date have not been definitive.

When the underground stress wave impinges on a structure, a very complex interaction of forces naturally occurs. If no deliberate isolation is provided, the gross motion resembles closely that of the adjacent free field, especially in terms of displacement. The higher frequency components of acceleration may be modified considerably. Forces on, and relative motion of, structural components are very dependent on many factors, including their stiffness and mass.

Instrumentation for measurement of structural loading and response resembles that for the free field. On massive structure walls, a stress gage flush with the wall measures the loading at that point satisfactorily--whether it can be taken to represent an average loading depends on conditions. In the case of flexible structures, the available gages represent an undue discontinuity in the wall.

Structural response can be measured by a combination of accelerometers, etc., and strain or relative displacement gages. In this connection, use has been made of scratch gages measuring peak relative displacement as back-up to dynamic instrumentation with considerable success.

Another type of gage being used fairly recently, both for structural motion and free-field motion, is a reed gage showing the shock spectrum of the motion to which it is subjected. Since this is a type of scratch gage, it must be recovered to obtain the data, which limits its usefulness in free field studies, but its results are directly meaningful to the structural engineer. Similar results can be obtained from other recordings only by rather complex analyses.

An important factor in the development of prediction methods is the knowledge of the characteristics of the medium. Some of these can be obtained from static tests on soil samples, but they do not complete the picture--dynamic characteristics are needed. Others can be calculated from full-scale test results, but they may be ambiguous and in any case apply only to the test region. Seismic measurements are a relatively cheap and easy method for measuring some elastic characteristics, but they measure only the initial modulus, for instance, which may be somewhat different from that at high stress levels. They have usefulness however, which would be improved by reliable methods of deriving shear modulus or Poisson's ratio.

In summary, then, the problems of instrumentation in the underground area are only reasonably under control. The capability exists for obtaining much useful data, but there are several definite needs. Without much regard for the feasibility of filling them, they include:

1. A free-field stress gage which can, in effect, be installed without disturbing the medium.

2. A structural loading gage which can be applied to flexible structures without creating an undue disturbance to the structure or the fill.

3. A particle velocity gage with good low-frequency response and adequate range.

4. A particle displacement gage of the same specifications.

5. A soil strain gage which can be installed at depths in a small hole with good assurance of proper connection to the undisturbed soil.

6. Techniques for adequate measurement of dynamic characteristics of soils, to be used on samples and/or in situ, covering stress ranges of interest to protective construction.

Those involved in the field are not ignoring these needs. Work is in progress or proposed on all of them, I believe, with the possible exception of the first, and the responsible agencies have indicated their interest in providing the necessary support.

I am sure that in this short presentation I have omitted several items of interest or importance, and corrections, additions, or questions are in order. Thank you.

DISCUSSION

MR. E. G. PETERSON: One of our previous discussors mentioned some wind velocities that were in the crater and the vicinity thereof in a one megaton burst. I think it was 4,000 knots. Now, maybe they haven't experienced that, but what effect does that have on instrumentation measurements? You said you had some immediately in the bottom of the crater.

MR. SWIFT: I believe the figure given was 400, not 4,000. That is still high. Obviously, the blast is going to carry the soil away in the heart of the crater. There is a very serious problem in mounting air pressure gages so they will survive in this region. It is a problem particularly to make sure that the cable itself is not thrown away, but this is solved by using high-pressure cable and bringing it in deep enough underground. The air pressure gages must of necessity not be overtaxed if they are to survive. There is some question as to the validity of the measurements at this velocity. However, if the baffles are made large enough and the air duct is taken at right angles, it should measure the static overpressure.

MR. BOB MRAS (Planning Research Corporation): What was the band on your channels for carrying your information?

MR. SWIFT: Well, of course, this is a generalization. The preponderant majority of them are zero to 300 cps. That is not only ours, but those used in the field. In a few cases, it is necessary to go to higher frequencies in response. Of course, with piezoelectric types of gages, it is necessary to sacrifice the low frequency and go to 10,000 cps. The preponderant majority have been found satisfactory at from zero to 300 cps.

DEVELOPING A PIEZO-ELECTRIC STRESS GAGE

W. E. Schmid
Princeton University

THE PIEZO-ELECTRIC EFFECT

If a crystal of some material like Rochelle salt, barium titanate or quartz is cut normal to certain axes, the opposite faces of the crystal slab thus obtained will develop a difference in potential when they are subjected to a normal stress. This phenomenon is called the piezo-electric effect. The equations which govern this piezo-electric behavior are:

$$-\bar{\epsilon} = s\,\bar{\sigma} + d\bar{E}_f$$

$$\bar{P} = d\,\bar{\sigma} + kE_f$$

$$\bar{i}_d = \frac{1}{4\pi}\frac{\partial \bar{E}_f}{\partial t} + \frac{\partial \bar{P}}{\partial t}$$

$$\frac{\partial^2 \bar{\xi}}{\partial x^2} = \frac{1}{V^2_c}\,\frac{\partial^2 \bar{\xi}}{\partial t^2}$$

The symbols used denote the following:

ϵ = normal strain　　E_f = field intensity

σ = normal stress　　P = polarization

s = elastic constant　　k = susceptibility

d = piezo-electric constant　　i_d = current density

$V_c = \sqrt{\dfrac{1}{\rho_c s}}$ = velocity of wave propogation

ρ_c = density of crystal　　ξ = displacement in direction x

and x and t are coordinates of space and time. The bar over a quantity denotes that it is a function of time.

Thus, if such a crystal slab is subjected to a normal stress, it will exhibit strain as well as a difference in potential between the two parallel faces (Fig. 1). This potential difference may be picked up, amplified and used to determine the stress which is exerted through the crystal at any instant of time.

Conversely, if a cyclical potential difference is impressed upon the opposite faces of such a crystal, the slab becomes strained and the crystal will start to vibrate with the frequency of the applied potential difference. This effect is called electrostriction and may be used to generate stress waves of known frequency and intensity. If these stress waves are measured at a distant point, their attenuation may be determined.

THE MEASUREMENT OF STRESS

Most conventional measurements of stress in a body, be they resistance measurements or any other type, are actually measurements of strain multiplied by a modulus of elasticity or a modulus of compliance. Such methods of stress determination are possible and relatively simple for static stresses if the body is linearly elastic. For transient stresses, the dynamic modulus of compliance has to be properly determined. Difficulties and uncertainties arise if the material exhibits nonlinear elasticity or visco-elastic behavior and if the stresses are of a transient nature with high frequencies.

The ideal stress gage is a gage which (a) measures stress as directly as possible, (b) has an instantaneous response, (c) has as little mass and as few moving or straining parts as possible, and (d) has the same stress-strain characteristics as the material itself.

It is obviously most difficult to develop an ideal stress gage which satisfies all these conditions simultaneously. And because of requirement (d), there can be no unique ideal stress gage for all materials but only, at best, a good gage for a particular type of material. It is our opinion that in rocks, and possibly soils, the utilization of the piezo-electric effect for transient stress measurements may come close to meeting the ideal requirements as stated above.

Although the piezo-electric effect has been known for several decades and has been used extensively in the communication industry, its utilization for the measurement of stresses is a relatively recent development. One reason for this seems to be that the profession has been preoccupied with the measurement of static stresses. The piezo-electric effect does not lend itself to the measurement of static stresses nor to that of transient stress waves with less than 5 to 10 cps, because the electric charge generated leaks away either across the piezo-electric crystal or across the amplifying tube. But for higher frequencies and with the proper choice of crystals, a piezo-electric stress gage may come very close to the ideal conditions.

FIELD STRESSES AT A POINT

The measurement of stress at a point due to a dynamic disturbance somewhere else allows us to determine the attenuation as well as the propagation velocity of the resulting stress--or shock wave--if the magnitude and time of the disturbance are known. If neither of these is known, we still may determine these quantities if we measure the synchronised stresses at two distant points. Thus, for the design of protective underground structures, the measurement of field stresses becomes very important. Since a complex spectrum of pressure, shear and surface waves results from a single disturbance at a point, the complete state of stress must be determined at any instant of time. Stress at a point is a tensor quantity having 9 components, 6 of which are independent.

The normal stress σ on a plane normal to the direction "N" is given by the following equation:

$$\sigma N = \sigma x l^2 + \sigma y m^2 + \sigma z n^2 + 2 \tau xy\ lm + 2\ \tau\ xz\ ln + 2\ \tau\ yz\ mn \qquad (2)$$

where l, m, n are the direction cosines of N to the x, y and z directions, and the σ's and τ's are the respective normal and shear stresses.

If the directions x, y, z are the principal directions, the shear stresses are zero and Equation (2) reduces to:

$$\sigma N = \sigma x l^2 + \sigma y m^2 + \sigma z n^2$$

Looking at an arbitrary cartesian coordinate system "A" with axes 1, 2, 3, we may designate the direction cosines of the direction 1 to the x, y, z directions by l_A, m_A and n_A. The direction cosines of the remaining directions 2 and 3 may be obtained by a cyclical shift.

	x	y	z
1 :	l_A	m_A	n_A
2 :	m_A	n_A	l_A
3 :	n_A	l_A	m_A

The same applies for a second coordinate system "B" with the directions 4, 5, 6 and the direction cosines l_B, m_B and n_B (Fig. 2).

4 :	l_B	m_B	n_B
5 :	m_B	n_B	l_B
6 :	n_B	l_B	m_B

If we now measure the stress in three orthogonal directions 1, 2, 3 and again in the directions 4, 5, 6, we can establish the previous relationship for each of the six directions giving:

$$\begin{aligned}
\sigma_1 &= \sigma_x l_A^2 \quad \sigma_y m_A^2 \quad \sigma_z n_A^2 \\
\sigma_2 &= \sigma_x m_A^2 \quad \sigma_y n_A^2 \quad \sigma_z l_A^2 \\
\sigma_3 &= \sigma_x n_A^2 \quad \sigma_y l_A^2 \quad \sigma_z m_A^2 \\
\sigma_4 &= \sigma_x l_B^2 \quad \sigma_y m_B^2 \quad \sigma_z n_B^2 \\
\sigma_5 &= \sigma_x m_B^2 \quad \sigma_y n_B^2 \quad \sigma_z l_B^2 \\
\sigma_6 &= \sigma_x n_B^2 \quad \sigma_y l_B^2 \quad \sigma_z m_B^2
\end{aligned} \tag{3}$$

Assuming the orientation of system "B" with respect to system "A" is known, we may write:

$$\begin{aligned}
l_B &= f_1\ (l_A) \\
m_B &= f_2\ (m_A) \\
n_B &= f_3\ (n_A)
\end{aligned} \tag{4a}$$

Furthermore, the following relations must hold:

$$\begin{aligned}
l_A^2 + m_A^2 + n_A^2 &= 1 \\
l_B^2 + m_B^2 + n_B^2 &= 1
\end{aligned} \tag{4b}$$

Hence, the measurement of the six components of stress σ_1 through σ_6 allows the determination of the complete state of stress at a point with the help of Equations (3) and (4) at any instant of time. For this purpose then, a synchronized time record of the signal from each of 6 piezo-electric gages becomes necessary.

PRACTICAL CONSIDERATIONS

Since the period of the stress cycles may be not more than a few milliseconds, the recording apparatus must be responsive to pulses of extremely short duration. This practically eliminates mechanical recorders because of inertia effects. On the other hand, field equipment should be relatively rugged, simple and light. Therefore, at the moment it is considered best to record the amplified signal from the crystals on magnetic tapes or drums. These magnetic tapes can then be used directly to provide the input to an electrical analog computer which determines the magnitude and direction of the principal stresses σ_x, σ_y, σ_z according to Equations (3) and (4) (Fig. 3). This procedure has the additional advantage that the input speed may be adjusted to the optimum operating frequency for the analog computer.

DETAILS

Figure 4 shows the details of a piezo-electric pressure gage used by the British Road Research Laboratory to measure dynamic stresses in compacted earth fills under moving loads. The sensitivity of this gage was $\pm$.03 psi with a range of 150 psi. It had a waterproof insulation for 10,000 meg ohms. The signal was picked up and put on an oscilloscope and recorded by a camera. Figure 5 shows the tentative design of a pilot gage to be used in laboratory and field tests at Princeton University.

Schemes for arranging the two orthogonal coordinate systems "A" and "B" along whose axes the six stress components are measured are shown in Figs. 6a and 6b. Figure 6a would be at a point where there is free access for installation of the gages; Fig. 6b is for the installation of gages at the bottom of a drill hole.

In conclusion, it may be stated that there is an urgent need for the development of a reliable and rugged, yet simple and responsive, dynamic stress gage. We hope and believe that a piezo-electric gage may meet these exacting requirements.

DISCUSSION

COMMANDER CHRISTENSON: I was wondering how you calibrate that last gauge since you apparently cannot apply a static pressure to it.

DR. SCHMID: Well, you can always calibrate it by dynamic pressure. You could calibrate it, for example, resting it in a cylinder in which you induce vibrations. You can, of course, also correlate that by theoretical computation, getting the stress from the piezoelectric constant.

PROFESSOR RINEHART: Are there any more questions?

MR. T.L. WHITE: I am interested in the frequency response of your analog computer.

DR. SCHMID: I am afraid I cannot give you the answer as yet because we are just in the development stage of it. That is a very good question. It will certainly have to be looked into very carefully.

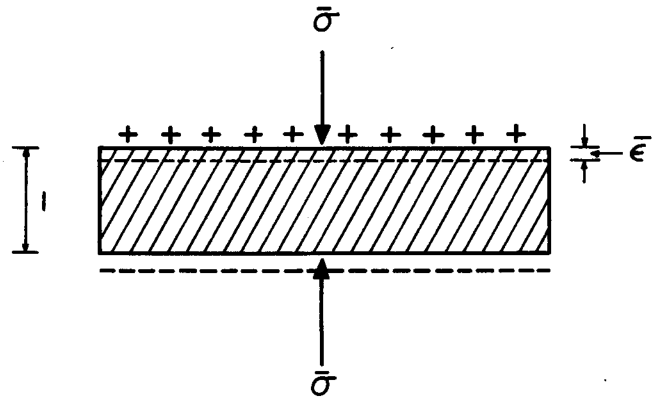
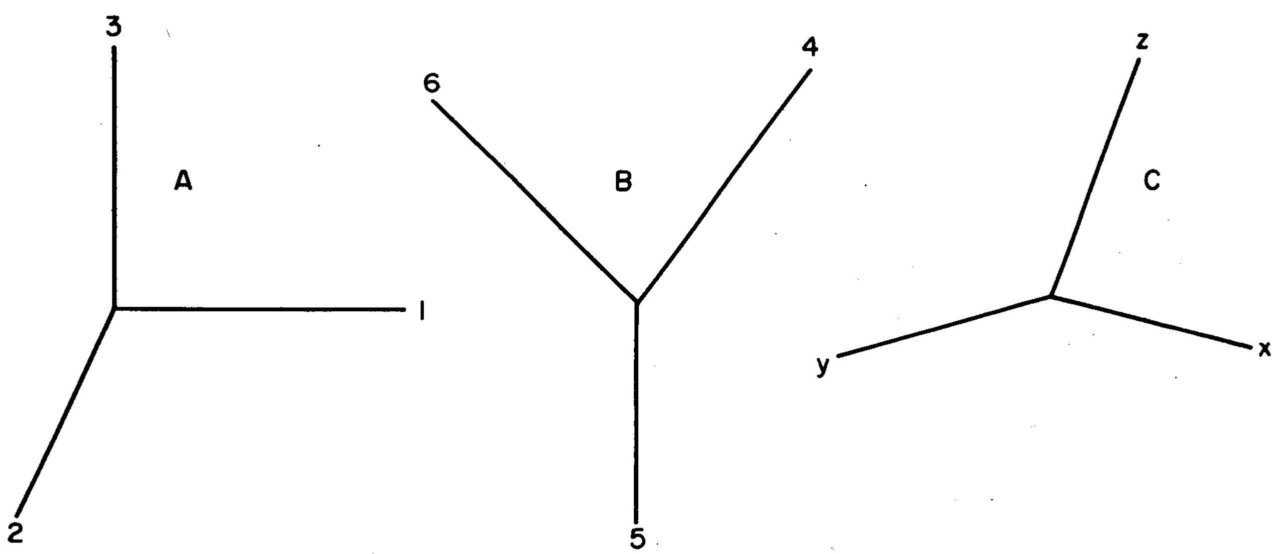


Fig. 1 — Piezo-electric crystal under transient stress



A and B are fixed systems along whose axes the normal stresses σ_i are measured to give the orientation of the floating system C i.e. the principal directions x, y and z

Fig. 2 — Orthogonal coordinate systems A, B and C

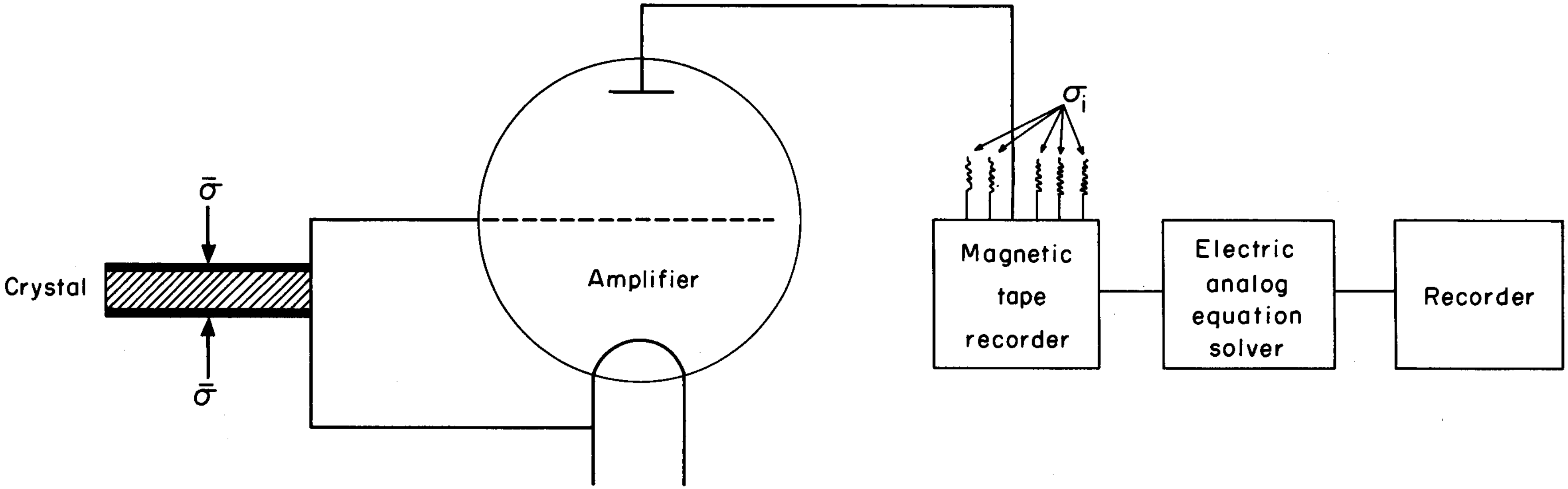


Fig. 3 — Schematic diagram of stress gauge and data evaluation

Diameter:	3.00 in.
Thickness:	0.645 in.
Thickness of crystals:	0.50 in.
Compression:	6.45×10^{-8} in/lb/psi

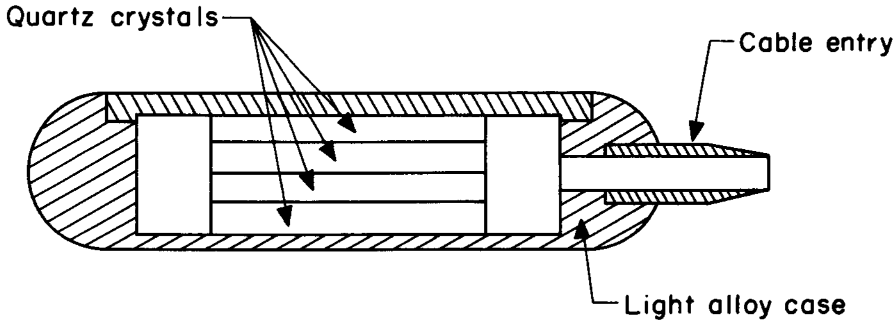


Fig. 4 — Detail of Piezo-electric stress gauge

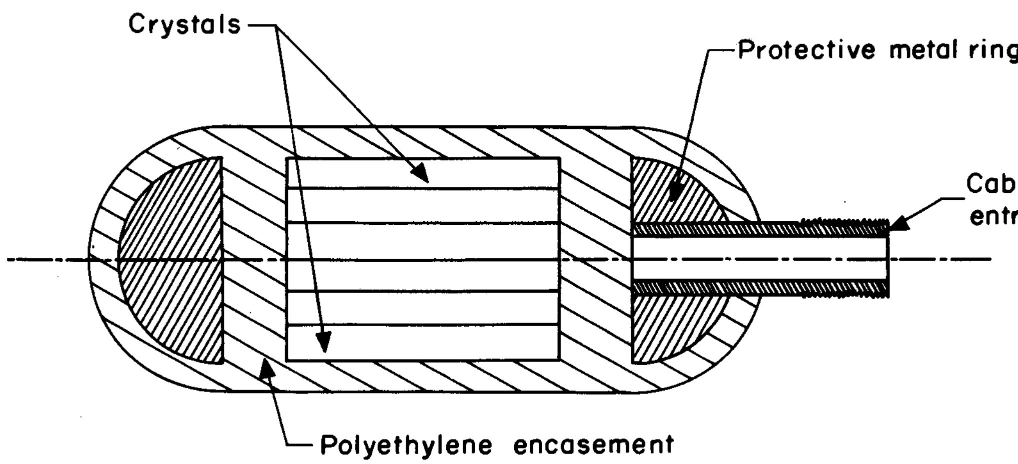


Fig. 5 — Cross section through gauge

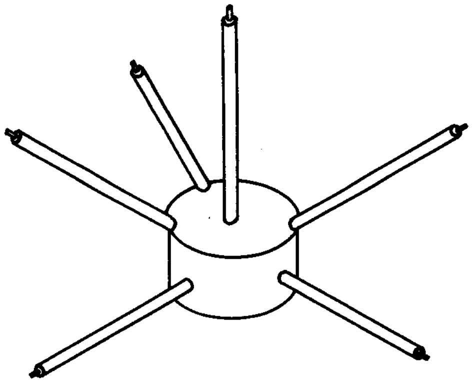


Fig. 6a—Stress measurement; free access to gages

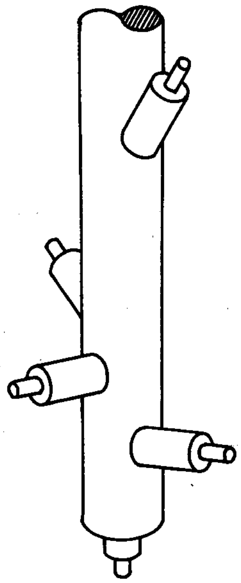


Fig. 6b—Stress measurement; gages in drill hole

SOME FACTORS TO CONSIDER IN SITE SELECTION AND DESIGN OF UNDERGROUND PROTECTIVE STRUCTURES

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Note: This paper was not presented at the Second Protective Construction Symposium at Santa Monica, California, March 24 to 26, 1959; but is submitted on request for inclusion in the Proceedings of that Conference.

It has been generally agreed that in event of an attack upon this Nation by a foreign power, certain critical facilities and personnel would need to be protected by being housed in underground structures of such depth as to avoid destruction by nuclear explosions. Since the United States is a very large nation and critical facilities are located at many points where topography and geology may vary considerably, conditions affecting site selection and design of protective structures may also vary widely.

Where a critical facility may be located in or near mountainous terrain it would seem that the problems and expense of providing underground shelter would be much less than where the facility would be located in level territory. In either case however, the problem of finding a suitable body of rock in which to construct the shelter may require considerable investigation.

In mountainous areas the shelter could probably be located at or above the level of surrounding ground. This would reduce problems of entry and supply, and drainage and ventilation. Level or nearly level roadways could allow direct transfer of supplies and equipment from trucks. Ground water, condensate, and liquid wastes could flow out by gravity.

Power requirements for necessary services would be least in this type of installation. When a facility is located in a flat area, shelter would have to be provided below grade, and the problems of supply, entry, and drainage would be much more difficult. Much more power would be required for operating elevators, pumps, etc. In addition, it is possible that gas problems, viz., ethane, methane, sulfur dioxide, etc., could be more acute in this type of shelter. While it is likely that air conditioning might be employed, and a slight increase in air pressure above atmospheric used, the permissible pressure for an extended period would have to be very low and could not be high enough to resist infiltration of noxious gases or ground water, at the levels necessary for protection.

Some speakers discussed location of underground shelters in large intact masses of rock, and described design and construction of shelters having floor layouts similar to mines, for reasons of speed in excavation and reduction in problems of roof support.

The suggestions are good if such areas can be found, with the possible exception that intersection points of corridors or tunnels should be made as small as possible. Otherwise higher rock stresses could occur at these points, both in the roof and the pillars. Actually, it is not likely that large intact masses of rock will exist at suitable depth within reasonably accessible distance of every facility needing protection.

Depth should be kept to the minimum required for safety, because of depth problems, only some of which are mentioned above.

It is probable that few, if any, ideal conditions will be found at any suitable location. Nearly all rock will have fissures, joints, bedding planes, etc., along which movement may occur and water may travel. Any plans made should contemplate the existence of such mechanical defects in the rock as well as some possible chemical alterations which are also not uncommon.

Natural rock at depth is usually under stress, and plastic movement due to release of stress may result in the development of cracks and spalling over a period of time. This condition is frequently found in tunnels where the excavation period extends for a year or more. Several examples of rock excavation were described in which roof bolts held the rock in place, and speakers told of tunnels where no supports of any kind were used. It should be remembered that these excavations are under static load, whereas the protective structures under consideration here may be subjected to severe dynamic loads. Rock is heavy, and even small pieces falling can be very dangerous. Damage resulting from falling rock could be disastrous, At best it could reduce the usefulness of the shelter, and might result in serious damage and loss of life, and even necessitate abandonment of the shelter at a time when it would be vitally needed. Consideration of costs of constructing, equipping and maintaining protective shelters underground should indicate that suitable roof supports would not be a major item of cost, and the risks involved in omitting them may not be worth the relatively small saving. If the shelters are built they should be made as safe as is possible.

The shock wave resulting from a nuclear explosion would travel some distance through the ground, and undoubtedly would be noticeable at the protective structure. It may not be practicable or even possible to build shelters completely beyond range of the shock wave. It is more realistic to consider their location as somewhere beyond the dangerous region. It is probable that the wave would travel farther through hard rock than through softer rock or soft ground. The important thing is to be able to absorb the shock without damage.

Therefore, some resilient material for support would seem to offer better protection than a more brittle material. Steel would most likely perform better than any other structural material.

Another important element of the protective shelter is the access corridors. These may be tunnels, where the shelter is in mountainous territory, or shafts where the shelter is located in flat country. In either case several corridors at various locations should be provided to avoid the possibility of the shelter being cut off from the outside by collapse of the communication corridor. These tunnels or shafts would have to penetrate zones of soft ground and unstable and weathered rock, any of which could be wet, and would require adequate support to prevent collapse under normal conditions as well as under attack conditions. Their locations would have to be such that if one or more were destroyed or seriously damaged, others would be available for escape from or service to the shelter.

Remaining in a confined space under unaccustomed conditions would be a very great strain on many people, and the strain would be greatly exaggerated by a knowledge or suspicion that good and adequate roof support had not been installed. Any rock fall, however slight, could create uncertainty and even panic among those in the shelter. Under conditions of threatened attack, anxieties would be great, nerves would be tense, consequently protective shelters should be such as to inspire confidence. This should be kept in mind when considering design for such structures.

Insofar as actual design is concerned, information would be needed on anticipated shock loads, minimum and maximum limits of room width, the minimum required wall and ceiling heights, and the total floor space required. With this information the writer could make suggestions for roof and wall support for any reasonable rock conditions.

Supports for access corridors could also be developed when size and load requirements were known. Ground conditions for the corridors would have to be assumed pending selection of an actual site.

FIGURES

1. Small tunnel supported with steel liner plates. Tunnels may be of various cross sections and sizes with this type of support. Also, vertical shafts and inclined tunnels commonly employ these plates for holding the surrounding ground. Grout, pea gravel, or other material is pumped behind the plates to fill any open spaces which might be caused by the excavation operation.

2. Small tunnel, approximately 11 by 11 ft, using steel liner plate support. Note staggering of end joints and the butted end joints which transmit membrane forces effectively.

3. Sewer tunnel with liner plates and H-beam ribs. Note the monorails which carry 10-ton loads on a moving crane, indicating the great strength of this type of support.

4. Double lane highway tunnel in massive broken rock. Support consists of 10-in. WF beams curved to semicircular shape resting on sill and short posts with steel liner plates between ribs. Note that length of liner plates parallels axis of tunnel.

5. Double lane highway tunnel in thinly bedded sedimentary rock. Support consists of 8-in. WF beams with lagging spanning the spaces between ribs. This tunnel was later lined with concrete. Width is approximately 32 ft. Steel is capable of resisting heavy shocks of explosions as the heading is blasted to advance the tunnel.

Fig. 1

Fig. 2

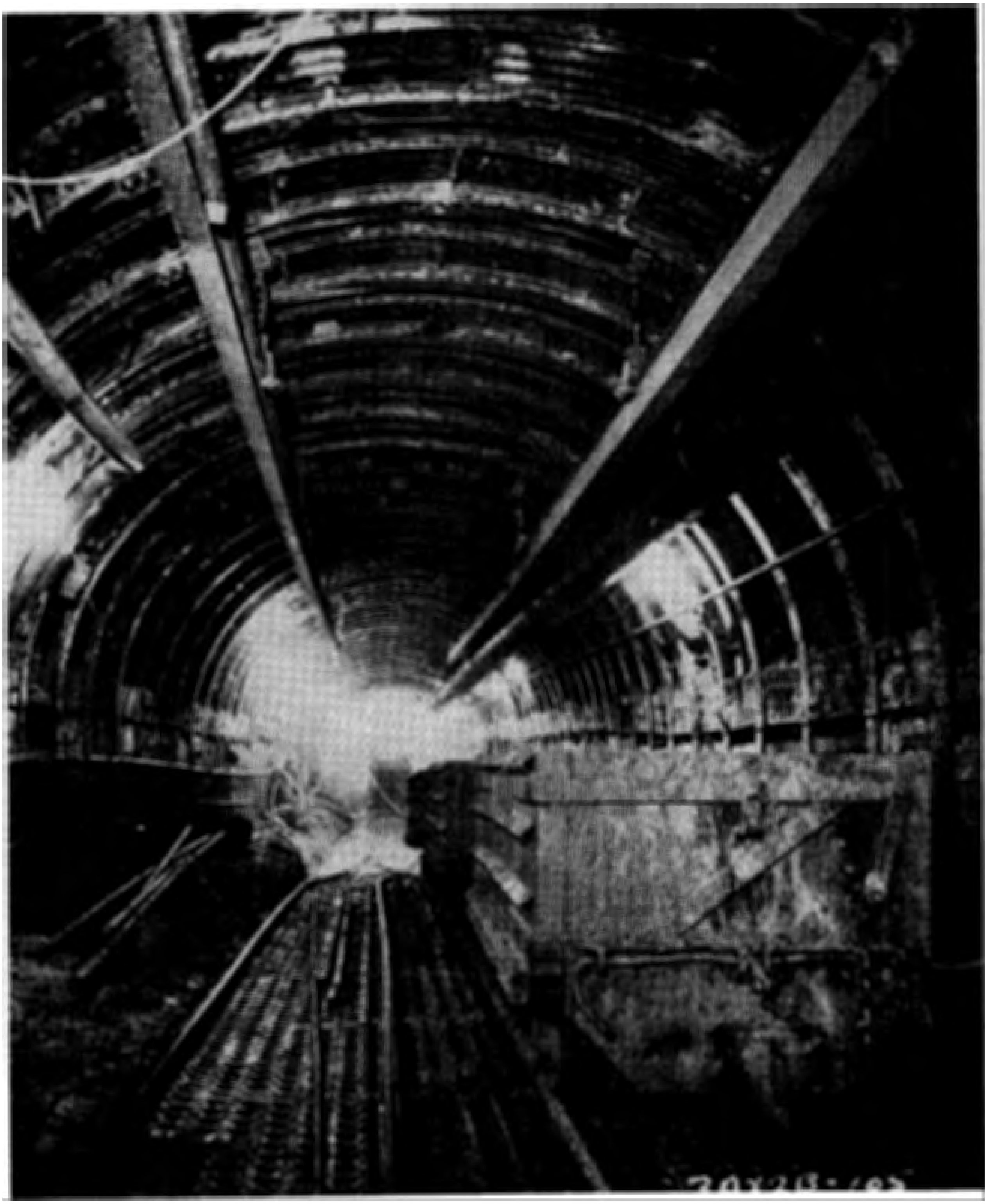

Fig. 3

Fig. 4

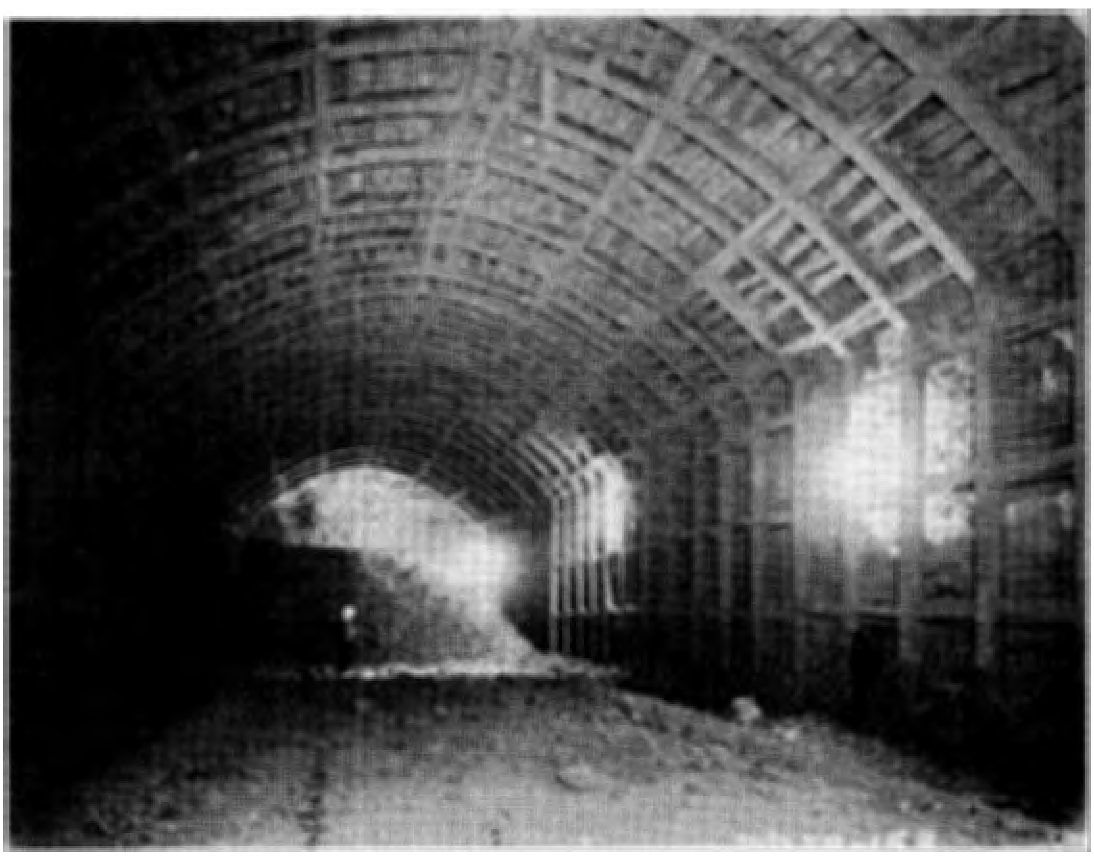

Fig. 5

FACTORS TO BE UNDERSTOOD IN SITE SELECTION

M. D. Kirkpatrick
Chief, Protective Construction Branch
Office, Chief of Engineers, Department of the Army

The subject for discussion this morning is site selection for deep underground construction. Six presentations have been arranged, as shown on your program. I will open the session with a discussion of a number of factors found to be particularly significant based upon experience of the Corps of Engineers during many years of investigation, study, planning, and construction in this field.

PLANNING FACTORS

Site selection for a deep underground installation must include consideration of a great many factors not usually involved in the selection of sites for conventional facilities. These further add to the difficulty in finding a completely acceptable site. They are very likely to be outweighed by other considerations unless those responsible for planning and guiding the site selection are fully cognizant of their implications. (Fig. 1.)

Careful appraisal of the many requirements which generally develop in connection with site selection is of highest importance. A glance at Fig. 1 should furnish a graphic illustration of the many--and sometimes contradictory--requirements involved. I have purposely jumbled these items to better illustrate the nature of the initial problems facing the planner. They will be placed in a more orderly sequence, later. You will note the frequent dollar signs in this figure. They are not there to emphasize additional cost for a deep underground facility--we must expect to pay more for the additional protection we gain by going underground--but they are there as reminders that seemingly insignificant requirements which are taken for granted in conventional facilities will usually increase the cost of several different features when placed underground. I have in mind such items as space requirements--for equipment, for vehicular traffic, for operations which are not essential to the functioning of the facility in time of emergency, and for dining and sleeping facilities which can and should be on a far more austere basis than normally expected in conventional facilities. Additional rock excavation, power, air conditioning, water storage, and other features will be required in addition to the space requirements, alone.

FACTORS INFLUENCING DECISION

The decision to plan for--or to construct--a deep underground military installation may have to be made long before specific site selection can be accomplished. Figure 2 illustrates some of the possible steps in this planning, i.e., analysis of the function, mission, and importance of the facility, evaluation of all other means of assuring reasonable chances of survival, the justification for using deep underground construction (in lieu of other types), and finally an indication of desired protection (depth of cover, blast pressure, chemical, biological, fallout, or other requirements). Under current Department of Defense instructions, the degree of protection must be based upon a target analysis using anticipated enemy capabilities as to weapons and delivery systems. I might add that the "justification" at the end of this stage may be only in the mind of the Service, Agency, or Command responsible for the installation. Each project must be carefully worked into the overall plan of the Department of Defense. Evaluation of the factors shown on Fig. 2 and the decisions reached after their evaluation can have a profound effect upon the choice of sites, as well as cost, construction difficulties and ultimate effectiveness of the installation when completed. It is highly essential that during these initial planning stages, maximum use be made of experts in such specialized fields as weapon effects, vulnerability, probabilities, protective design, mining technology, geology and interpretation of geologic conditions, plant layout, engineering, design, and construction.

RELATION OF PLANNING FACTORS TO CHOICE OF SITES

Figure 3 shows an illustrative example of an underground site. The requirements listed at the bottom of this figure are representative of those which might be furnished to those charged with the selection and recommendation of sites based upon decisions reached as a result of the initial planning. All of these, of course, will not be applicable to each or to all types of installations. I have prepared this figure in order to emphasize a few of the factors, indicated by asterisks(*), which warrant very careful consideration, during the planning as well as during the site selection phases.

First, depth of rock cover. It is very easy to specify hundreds of feet of cover in the interest of greater protection. It is not easy to find sites which will satisfy this condition. The length of horizontal access tunnels may increase several hundred feet for each additional 100 ft of rock cover attained. This could add many thousands of dollars to the cost. And it may not result in more than a small increase in overall survival probability when the relative vulnerability distances for both rock damage and air blast damage are carefully weighed. I may be challenged on my statement about increased length of access tunnels for additional cover. However, when we consider certain inevitable requirements such as access, proximity to airfields and base facilities, and possibly certain special geographical or tactical requirements for the installation, I am sure you will admit that we will not be able to site very many installations in locations such as the Grand Canyon, The Royal Gorge, or other areas where almost vertical cliffs may be found.

A second factor which warrants special consideration is that of space requirements. Experience has shown that the total cost of providing space in new underground installations may be from two to four times the cost of providing space in unprotected buildings of conventional construction. We can, therefore, get a much greater return for these extra dollars if we limit the facilities, space, and special accommodations required to be provided underground to only those of critical operational essentiality.

In connection with the investigation, consideration, and selection of specific sites, I want to emphasize, again, the necessity for a careful evaluation of the relation of cover and access. To attain maximum cover at a relatively flat site by use of steeply sloping access tunnels will add to the cost and time required for excavation. Drainage and utility maintenance problems as well as operational cost will be increased. At a 10 per cent grade 1,000 ft of access tunnel would be required to attain an additional 100 ft of cover, and this is a mighty steep grade. Six per cent will usually be limiting--this would require 1700 ft per 100 ft of depth. Deep vertical shaft entrances are costly and time consuming to construct, require elevators, are more vulnerable to being blocked and will, in many instances, seriously restrict construction and operational access. It is, therefore, desirable to seek terrain where access to the required depth of cover can be attained by means of relatively horizontal tunnels of minimum length. The requirement for and advantage of two access tunnels are, perhaps, obvious. To reduce the possibility of having both of these blocked by the same attack requires a reasonably large separation of the portals. A through-type tunnel such as illustrated will provide some advantages in relation to blast effects within the tunnel. This will usually require at least one of the access tunnels to be longer than the minimum necessary to reach the desired cover, and terrain should be chosen to best fit this requirement, if practicable.

CHECK LIST

Figure 4 shows a suggested check list of factors to be considered in connection with site investigations. These are grouped under two phases. First are those factors which should be considered and evaluated during the initial planning phase for the purpose of elimination and selection of those sites considered suitable for more detailed engineering investigations. Second are those factors which should be considered during the more detailed engineering studies upon which final recommendations would be based. Several important factors are indicated by asterisks(*). For example, consider the possibility of using an existing mine. Surveys of existing mines were conducted jointly by the Corps of Engineers and the U. S. Bureau of Mines from 1947 through 1952. Investigational, engineering, and cost studies were made by the firm of Guy B. Panero, Engineers, under contract with the Chief of Engineers in 1947 and 1948. These and subsequent studies and experience during the past ten years in connection with military planning and construction activities indicate that where existing mines are available and suitable for the required use, they could be adapted at great savings over the cost of new excavation. This saving could possibly amount to as much as one-fourth to one-third, based upon general order-of-magnitude costs for the rock excavation portion of a new underground installation.

Accessibility is an important factor, both from a construction as well as an operating standpoint. Significant costs may be incurred in providing access to remote sites, particularly if heavy vehicles or large volumes of traffic or rail facilities are required.

A source of water may prove to be a very critical factor. If a stream of dependable flow is close by or if wells of ample capacity can be drilled at the site, the requirement for protected storage of industrial or cooling water can be reduced. The dissipation of heat during the continued operation of a protected underground installation after loss of outside facilities has proven to be a difficult problem and can require costly underground space for cooling-water storage.

The necessity for support facilities should not be minimized. Availability of an existing base near the site will reduce the extent--and cost--of providing such facilities in connection with the underground installation. If proximity to an existing base is made a <u>requirement</u>, the latitude in choice of suitable sites may be narrowed considerably. In fact, almost any suitable site requiring a large amount of rock cover will likely require support facilities in addition to the primary underground installation.

Preliminary design studies should be made to confirm the adaptability of the facility to the terrain and to permit reliable cost estimates. Exploratory core drilling is also essential prior to final site selection, since it could very well indicate the necessity for modification of assumed excavation plans, facility re-arrangement to suit the rock structure, or the necessity for difficult and costly rock support.

The remaining factors on this check list require little explanation. It is obvious that no site will satisfy all of them. However, I cannot overemphasize the importance of good judgment in assigning relative weights to these factors appropriate to the mission of the facility--and the care with which each possible site is evaluated against these factors. Only by such a procedure will it be possible to make sound recommendations on final site selection.

SUMMARY

I hope this presentation has increased your understanding of a least some of the significant factors to be considered in connection with site selection for deep underground installations. In conclusion, I would like to emphasize the following five considerations, which I feel are of utmost importance:

1. Careful screening of space and protective requirements during the initial planning phase.

2. Appreciation of the relation of depth of cover, access, site conditions, functional requirements, utilities, and support facilities to the availability and choice of sites.

3. Desirability of using technical advice and experts in plant layout, engineering, geology, mining technology, weapon effects, and protective design during the initial planning and the site selection phases.

4. Necessity for exploratory core borings for each site considered and definitely prior to final choice of site.

5. Judgment in assessing the relative importance of the various requirements in terms of the operational mission and comparative evaluation of each potential site against each of these requirements as a basis for final selection.

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DISCUSSION

MR. PENTAGOFF (Los Angeles District, Corps of Engineers): Would you emphasize the importance of portals and tunnels in site selection?

MR. KIRKPATRICK: If you are referring to the condition shown in Fig. 3, I have tried to show the desirability of selecting a site where the topography is sufficiently steep and the geology such that tunnel portals may be developed at the required depth of cover without long sections of open cut or the added cost of constructing cut-and-cover concrete sections leading to the rock portal faces. If we cannot find a site which will permit developing portals at locations such as shown here, or if the entrances must be placed in the toe or flatter portions of the slopes where the material and geological structure will not permit development of a stable tunnel portal, unnecessary costs will be incurred. That brings up the importance of geological studies.

DR. JAMES ARMSTRONG: I think you made a statement that underground installations are always more expensive than surface installations. Do you mind saying from your experience what the factor is when the geology conditions are favorable?

MR. KIRKPATRICK: Two to four times, depending on the facility.

DR. ARMSTRONG: Two to four times?

MR. KIRKPATRICK: Yes, sir. This is a very difficult thing to answer. Everybody who read my paper before it was presented had a different answer. My two to four times factor is intended to indicate the increased costs incurred by going underground with a complete facility. You are going to build a communications center, let's say. If it is above ground we would usually build it on a base, and we would include the cost of the building, the equipment, a connection to the power line, water supply system, and a driveway from the road or street, etc. Now, that cost let us assume to be on the order of $15 per square foot for the building. The equipment might bring it up to $30 per square foot.

If we go into an underground site, it will not usually be built on a base. It may have to be in some rather remote area. We will most likely have to develop a water supply system. We will have to put in roads and site improvements. We have to excavate chambers in rock. We have to construct something within those chambers. By the time we are through, we will have spent for this facility anywhere from two to four times the cost of the conventional facility above ground.

No one wants to make the comparison on this basis, but to neglect the additional features which must be provided by going underground is risky. You have to pay for these facilities and this should be considered in planning. It is somewhere on the order of two to four times.

We have made planning studies in which we found we could adapt existing mines for general storage purposes for $10 to $12 per square foot. A storage warehouse building, above ground, may cost about $6 per square foot. In that case, the comparison is very low but it is still nearly two to one.

MR. ROBBINS (Burns & Roe): What about the ratio of time schedule?

MR. KIRKPATRICK: That's a good question. Rock excavation costs more than common excavation. It costs more partly because it takes more time to accomplish.

I think I pointed out that if shafts are required for entrances, the accessibility is limited and we must construct at least one entrance before we can accomplish much. If we have to excavate several hundred thousand square yards of rock from a site, it will probably take at least a year to do it. Until that rock excavation is accomplished, we construct the building or install equipment. A conventional facility could be started without waiting for the underground excavation. Even after the excavation is done additional time would be required above that required for a conventional facility. I would say the underground might take twice as long.

MR. NORTON (Geological Survey): We are talking about deep underground installations for protection against megaton weapons. Is a hundred feet the minimum we are considering? This is the figure you mentioned.

MR. KIRKPATRICK: No. I'm very sorry. I didn't mean to give the impression that a hundred feet was a minimum depth. I was stating additional lengths of tunnel required to attain an additional hundred feet of depth. Several hundreds of feet would be a more likely figure.

It would depend upon the size of megaton weapon, damage distances and probability of survival against a certain delivery system. The figure would generally be more than a hundred feet. I didn't mean that to be a limiting figure.

FIGURES

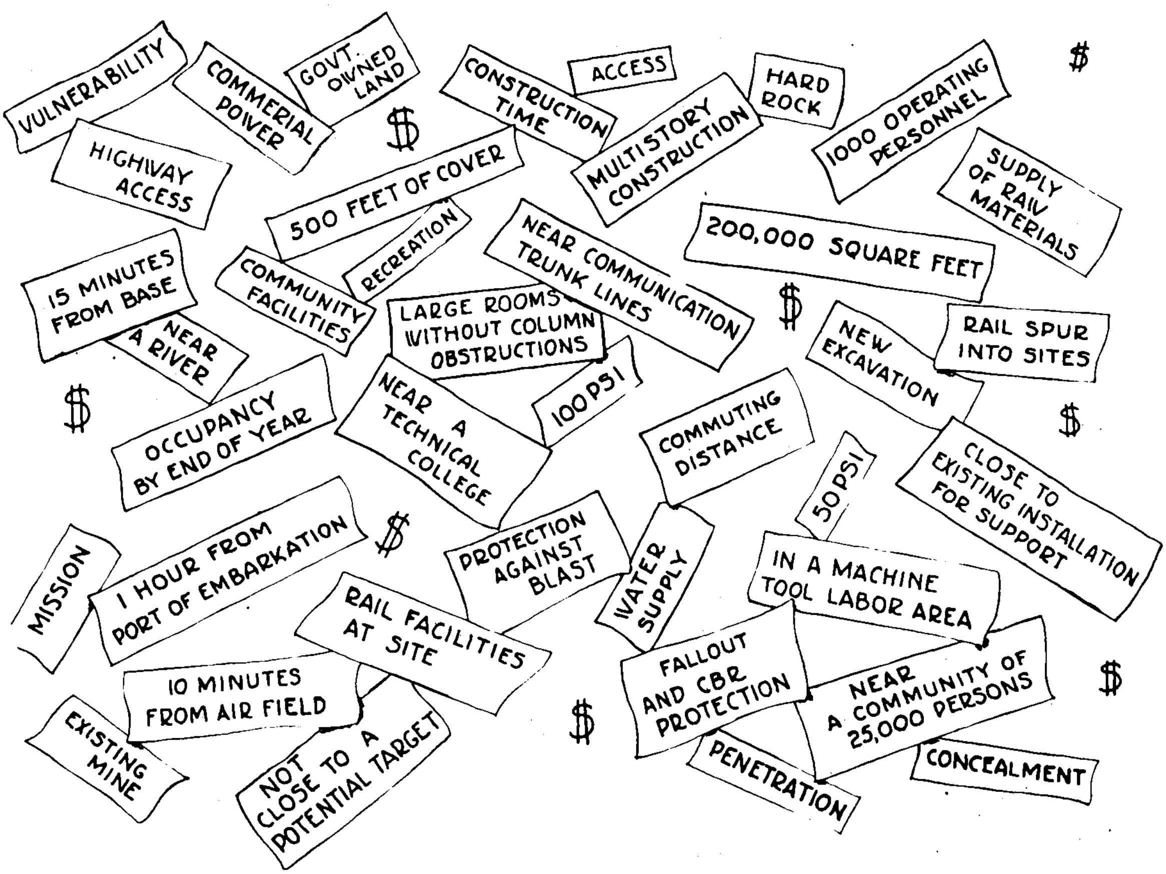
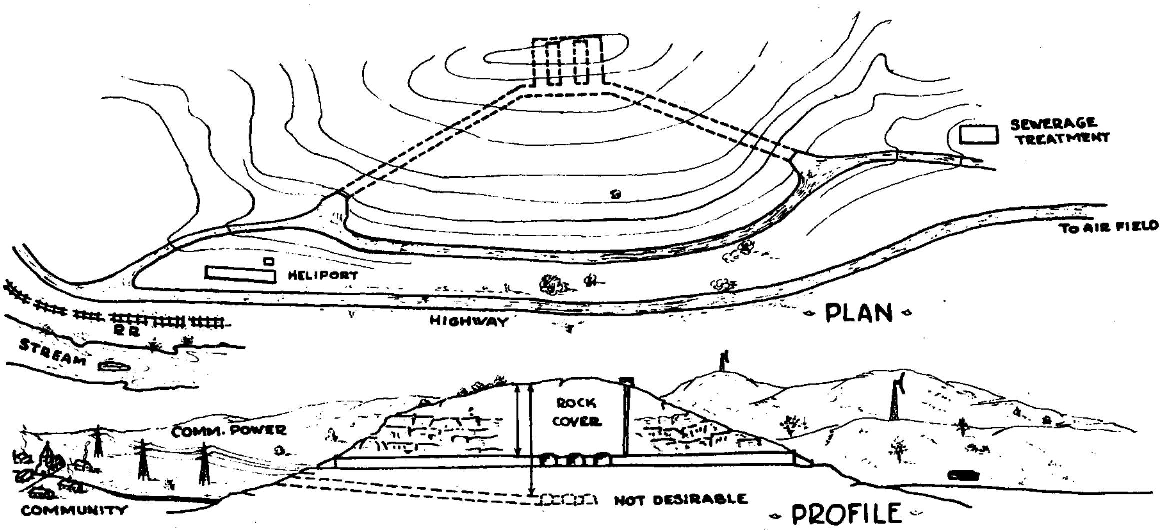

VULNERABILITY
COMMERIAL POWER
GOVT. OWNED LAND
$
CONSTRUCTION TIME
ACCESS
HARD ROCK
MULTISTORY CONSTRUCTION
1000 OPERATING PERSONNEL
$
HIGHWAY ACCESS
500 FEET OF COVER
SUPPLY OF RAW MATERIALS
RECREATION
NEAR COMMUNICATION TRUNK LINES
200,000 SQUARE FEET
15 MINUTES FROM BASE
COMMUNITY FACILITIES
LARGE ROOMS WITHOUT COLUMN OBSTRUCTIONS
$
NEW EXCAVATION
RAIL SPUR INTO SITES
NEAR A RIVER
$
100 PSI
OCCUPANCY BY END OF YEAR
NEAR A TECHNICAL COLLEGE
COMMUTING DISTANCE
$
50 PSI
CLOSE TO EXISTING INSTALLATION FOR SUPPORT
MISSION
1 HOUR FROM PORT OF EMBARKATION
$
PROTECTION AGAINST BLAST
WATER SUPPLY
IN A MACHINE TOOL LABOR AREA
RAIL FACILITIES AT SITE
10 MINUTES FROM AIR FIELD
FALLOUT AND CBR PROTECTION
NEAR A COMMUNITY OF 25,000 PERSONS
$
$
EXISTING MINE
NOT CLOSE TO A POTENTIAL TARGET
PENETRATION
CONCEALMENT

Fig. 1

TYPE OF FACILITY:	FACTORS INFLUENCING DECISION:	FACTORS INFLUENCING LEVEL OF PROTECTION:
COMMAND OR CONTROL	MISSION (Effect of loss)	TACTICAL IMPORTANCE
TACTICAL (Missiles, air defense)	VULNERABILITY (Will it be a target)	STRATEGIC IMPORTANCE
COMMUNICATIONS CENTERS	INSUFFICIENT DUPLICATION	ENEMY CAPABILITIES
PRODUCTION OR ASSEMBLY	DISPERSAL NOT ADEQUATE	GEOGRAPHICAL LOCATION
STORAGE (Reserves or operational)	PROXIMITY TO PRIME TARGET	CONTINUITY OF OPERATIONS

Fig. 2



REQUIREMENTS

*SPACE
- Operational
- Support
- Utilities
- Dining
- Sleeping
- Storage
- Medical
- Other

PROTECTION
- *Cover
- Blast
- Fallout
- Chemical
- Biological
- Warning
- Recovery

*ACCESSIBILITY
- Highway
- Railroad
- Airfield
- Supplies

DISPERSAL
- Industrial Centers
- Prime Targets

POPULATION
- Operating
- Maintenance
- Family

*SUPPORT
- Existing Base
- Community
- New Facilities

UTILITIES
- Power
- *Water
- Communications

FUNCTIONAL
- Tactical Reqts
- Radio Transmission
- Special Missions

Fig. 3

INITIAL INVESTIGATIONS	ENGINEERING INVESTIGATIONS
*POSSIBILITY OF EXISTING MINE	REAL ESTATE AND PROPERTY RIGHTS
AMOUNT OF COVER	ACCESS & UTILITY RIGHTS-OF-WAY
*ACCESSIBILITY	TOPO SURVEYS & MAPS
TOPOGRAPHY	*PRELIMINARY DESIGN STUDIES
GEOLOGY	ADAPTATION TO TERRAIN
EXISTING UTILITIES	*EXPLORATORY CORE DRILLING
*WATER	GEOLOGICAL MAPPING OF SITE
POWER	ANALYSIS OF CORES & SAMPLES
COMMUNICATIONS	AVAILABILITY OF WELL WATER
TRANSPORTATION	TESTS TO DETERMINE BLASTING CHARACTERISTICS
ADAPTABILITY TO SPECIAL MISSIONS	DRILLING TESTS
SITE DIFFICULITIES	DETERMINATION OF SPOIL AREAS
WEATHER & CLIMATE	AVAILABILITY & WAGE RATES FOR LOCAL LABOR
HYDROLOGY	LOCAL UTILITIES FOR CONSTRUCTION
GEOGRAPHICAL FACTORS	LOCAL MATERIALS & SUPPLIES
*SUPPORT FACILITIES	PREPARE REPORTS & COST ESTIMATES
COMMUNITY FACILITIES	CONSTRUCTION SCHEDULES & TIME
AIR FACILITIES	FINAL RECOMMENDATIONS

Fig. 4

ENGINEERING EXPERIENCES IN DEEP UNDERGROUND CONSTRUCTION AND THEIR EFFECT ON ENGINEERING DECISIONS

Walter S. Douglas
Parsons, Brinckerhoff, Hall and MacDonald

In considering how I might contribute to this Symposium, I was concerned that anything I might say would be redundant with what others would cover in a more expert fashion. Thus, it seemed to me best to present our experience on certain underground projects, and then to comment on various aspects of the over-all problem of developing secure and useful facilities underground. I shall also include some remarks on how one may reach design decisions in the light of the complex burgeoning technology being presented in this Symposium on the one hand, and remaining areas of uncertainty on the other.

I would begin by referring to certain photographs of a major underground excavation for a project for which we served as architect-engineers. Before introducing the pictures, however, I would emphasize that the rock encountered was a lineated greenstone which, from indications of extensive borings (and in fact as encountered in actual construction), was as good for this type of project as one is likely to encounter.

Figure 1 is a photograph of the excavation for a 32-ft wide access tunnel leading into the underground chambers. It serves to illustrate, as do the other photographs, the discontinuities in the form of joints and planes of weakness in rock formations, which are as significant and perhaps more significant in determining the behavior of rock along the periphery of an excavation than stresses which might prevail in a homogeneous elastic medium. You will note in the photograph that the tunnel is parallel to the strike of the lineation and therefore to the planes of weakness of the rock formation. On the left side of the photograph, the periphery of the excavation is perpendicular to the planes of weakness. Though the roof surface is very jagged, it is structurally sound and required a minimum of rock bolting. On the right side, however, the periphery of the excavation is parallel to the planes of weakness. This brings about a condition which, in the event there is a joint system perpendicular or at an angle to the planes of weakness, will produce large slabs that may fall into the excavated area. Since cross joints always exist to some extent, the possibility of dangerous slab falls is evident. For this reason you will note extensive rock bolting on the right-hand side of the tunnel roof to tie such slabs into rock above.

One important principle of underground construction is clearly evident from this photograph. Underground excavations should be oriented as nearly perpendicular to the strike of major joint systems or planes of weakness as possible. It is clear, for instance, that the roof of this tunnel would have been stable with much less rock bolting if the tunnel had been perpendicular to the alignment adopted. Actually, the tunnel gave access to chambers of much greater width which were perpendicular to it and therefore to the strike of the lineation of the rock. Thus, the wide chambers were oriented to best advantage, and the roof of the narrower access tunnel was adequately secured by roof bolts as illustrated in the photograph.

Figure 2 is another photograph taken of the same tunnel but from a different direction; the principles outlined above are clearly illustrated.

Figure 3 is a photograph of the upper one-third of a wide and deep chamber for which the lower two-thirds had not yet been excavated. The upper heading had been excavated by use of a full-face jumbo mounting horizontal pneumatic drills. Costs were typical for underground tunneling. The lower two-thirds, however, was excavated by the utilization of wagon drills in a fashion similar to some quarry operations and at much less cost than for ordinary tunneling operations. This is one of the reasons multi-story rock chambers can be excavated more economically than single-story chambers. The right-hand side of this photograph again illustrates a lineated rock formation and, in addition, shows the joints which cut across the planes of weakness and form the boundaries of potentially dangerous rock slabs.

Figure 4 is a photograph taken from approximately the same point as Fig. 3, but after the lower two-thirds of the chamber had been excavated. The unstable rock noted in Fig. 3 would have been in the upper right-hand corner of the roof of the finished chamber; however, much of that rock was removed. This illustrates another basic point in achieving stable roof conditions. Where massive slabs of rock are cut off in one direction by planes of weakness and in another direction by existing or potential joints, they should be scaled or blasted down. Rock bolts should not be used to support rock that appears about to fall. Such rock should be removed and rock bolts used to reinforce the most stable roof that can be created.

Figure 5 again illustrates rock discontinuities. On the lower right-hand side is a small slippage zone or fault which, because it is crossing a wall, is not dangerous. However, certain slabs in the roof which are clearly bounded by planes of weakness were blasted down before rock bolts were applied.

The photograph also illustrates <u>how not to</u> and <u>when not to</u> rock bolt. Rock bolting should be carried out as soon as possible after rock has been excavated and the roof has been scaled. In this case, because of certain contractual limitations and difficulties with funds, it was necessary to scale and rock bolt a roof some time after its original excavation. This procedure required the high platform shown in the photograph and is unwise from the point of view of both cost and roof stability.

Figure 6 is a photograph of a completed underground rock chamber with all potentially dangerous rock scaled down or otherwise removed and with the roof strengthened and supported by rock bolts on relatively close spacing. On the right-hand side of the photograph is the excavation for a cross chamber which was oriented at 90 deg to the main chamber. This cross chamber was the same width and height as the chamber illustrated. The long spans of the vaults created by the intersection of such wide chambers are particularly vulnerable to unstable rock conditions. Though no trouble was experienced at the intersection illustrated in this photograph, considerable rock at other intersections had to be removed above the theoretical pay-line in order to achieve stable roof conditions. For this reason we recommend that, if possible, intersections of wide underground rock chambers be avoided.

The completed chamber roofs illustrated in all the foregoing photographs have now stood approximately eight years. There have been no rock falls or indications of weakness.

In the light of our experiences on the project shown in these illustrations and on similar projects, we have formulated certain additional suggestions with respect to an over-all approach to the development of space and structures underground. Some of these principles can best be illustrated by referring to Fig. 7.

With respect to a roof support, it is our concept that at some distance above the zone of rock damaged by the blasting process is a stable, natural arch, much stronger than any that would or could be created by structural support. Where spans are reasonable--up to 45 ft or ever 50 ft --in good rock, the rock below the natural arch may be supported by rock bolts. Since the natural arch is nearer the periphery of the excavation at the haunches than at the crown, the rock bolts will generally be shorter at those points. You will observe in Fig. 7 how the length of the rock bolts increases progressively up to the center of excavation. It should be noted that, because of spalling that may be expected in the event of a shock, wire mesh should be attached to the rock bolts to prevent small pieces from falling.

Because of the immense strength of the natural arch above, the strength of steel in tension, and the compression introduced into the rock structure by the bolting process, we believe that rock roofs, up to the spans I have indicated, with rock bolts, will provide at least as much security as can be achieved with structural support from below. As a matter of fact, such structural support, if introduced, would be designed only to sustain the weight of rock between some natural rock arch above and the periphery of excavation.

Depending upon the rock encountered, however, at spans above 50 ft the length of rock bolts may become impractical and structural support may be required. This, of course, will substantially increase the cost of construction. It is clear, therefore, that the span of rock chambers should be established at the minimum consistent with effective and efficient utilization by the occupying agency.

Figure 7 illustrates the great economy to be achieved by multi-story construction. Certain elements of cost are fixed regardless of the number of stories--the relatively high cost of excavation of the initial adits, the cost of rock bolting and wire mesh, the cost of unusable space in the roof arch (although much of this is used for ducts and other utilities), and the cost of unusable space in the crawl space beneath the first floor (which is necessary if changes in utilities are to be made conveniently to respond to future needs). When the cost of such elements must be applied against a single story, the per-square-foot cost of useful space in underground construction is very high indeed. When fixed costs can be prorated against three floors, the cost per square foot of useful space is substantially reduced.

If this is so, one may wonder why four or even more stories would not be desirable. It seems to us that there must be some reasonable limitation to the height of unsupported rock walls and that good judgment indicates three stories to be the soundest approach. It should also be pointed out that three stories makes a very reasonable relationship between width and height of rock chamber and permits a compact economical arrangement for such mechanical installations as air conditioning, water distribution, electric distribution, plumbing, and the like.

Some further remarks may be in order with respect to the over-all configuration of underground installations. These I can best bring into focus by referring to Fig. 8, which is a hypothetical layout of an underground facility. At the lower part of the figure is the access tunnel, unobstructed by blast protective devices. The latter are provided in chambers at 90 deg to the access tunnel and should, therefore, not be exposed to such reflected pressures as would occur if they were at right angles to the advance of a shock wave. The portals of the two access tunnels should be as near 180 deg apart in orientation as possible.

The configuration illustrates a series of 45-ft chambers and one 50-ft chamber connected by 32-ft wide chambers, thus avoiding the long-span vaults at chamber intersections. The configuration also illustrates how requirements for greater chamber width for special purposes may be met. Note also the extensive excavation for water storage that may be required for both domestic use and industrial cooling during periods of isolation from outside utilities.

In the particular layout shown in Fig. 8, normal vehicular circulation is via the two-lane access tunnel from the right. The access tunnel from the left is adequate for a single lane of motor vehicles. It would, however, be used for that purpose only in emergencies. During normal use, it would be separated from the two-lane tunnel by an air barrier and would be used as the intake tunnel for air. Such an arrangement avoids the necessity for a separate air-intake shaft, which in some installations may cost as much as $300,000 to $500,000.

In closing, one further comment may be useful with respect to cost of underground construction as compared to cut-and-cover construction. Certain elements of an underground program are fixed, regardless of the extent of the useful space created. These include access tunnels, exhaust shafts, blast protective closures and devices, decontamination facilities, water supply, water storage for emergency periods when outside utilities are interrupted, a power plant, communication equipment, and kitchen and dining facilities. If costs of these necessary elements in an underground program are prorated against a small amount of useful space, the cost of an underground facility is very high indeed and may be so high that a more reasonable solution can be achieved by cut-and-cover methods. When the amount of useful space to be developed is extensive, however, the fixed costs, when prorated, do not become prohibitive and an underground installation becomes the desirable solution. Thus, it is impossible to generalize about the relative economy of underground rock construction and cut-and-cover construction. The proper solution will depend upon the extent of the useful space to be developed and the intensity of overpressure to be designed against.

DISCUSSION

CAPTAIN E. TATOM (NORAD): Mr. Douglas, did I understand you to say that mountain sites could not compete with the cut and cover at 200 psi?

MR. DOUGLAS: No, at twenty-five psi.

MR. BIEDERMAN: On these natural arches that you think are above the arch that is established, how much higher do you think these are?

MR. DOUGLAS: I think, for a roof spanning 45 feet, it may be ten to fifteen above, depending upon the rock itself.

MR. PFAFF (Burns and Roe): You say borings do not always tell the story. How extensive a test program would you undertake or recommend before locating the tunnel?

MR. DOUGLAS: Well, actually, that depends on the site. I can say that in two sites we have dealt with, two miles of borings have not been too much. I think it is important, as the design evolves, that the test borings be in two phases: A first phase to get a general idea of the site; then, if the site appears desirable, a second phase based on tentative layouts, including borings along proposed tunnel alignments (horizontal borings which can be carried up to 2,200 feet), and some vertically-inclined downward borings, which permit the use of the bore-hole camera. This camera will be described today. It is immensely useful in determining the orientation of the joints, plane of weakness, etc.

FIGURES

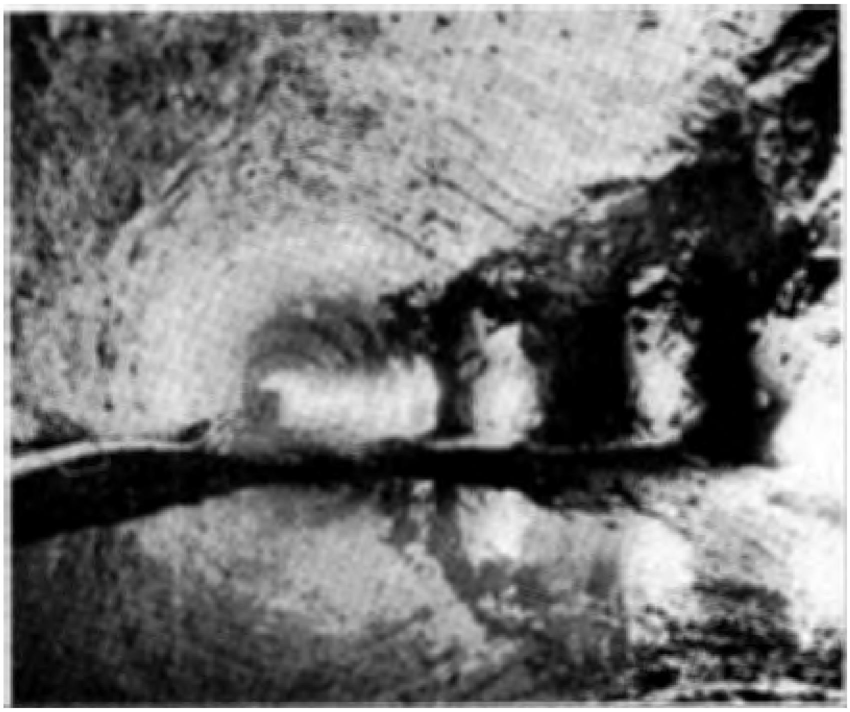


Fig. 1

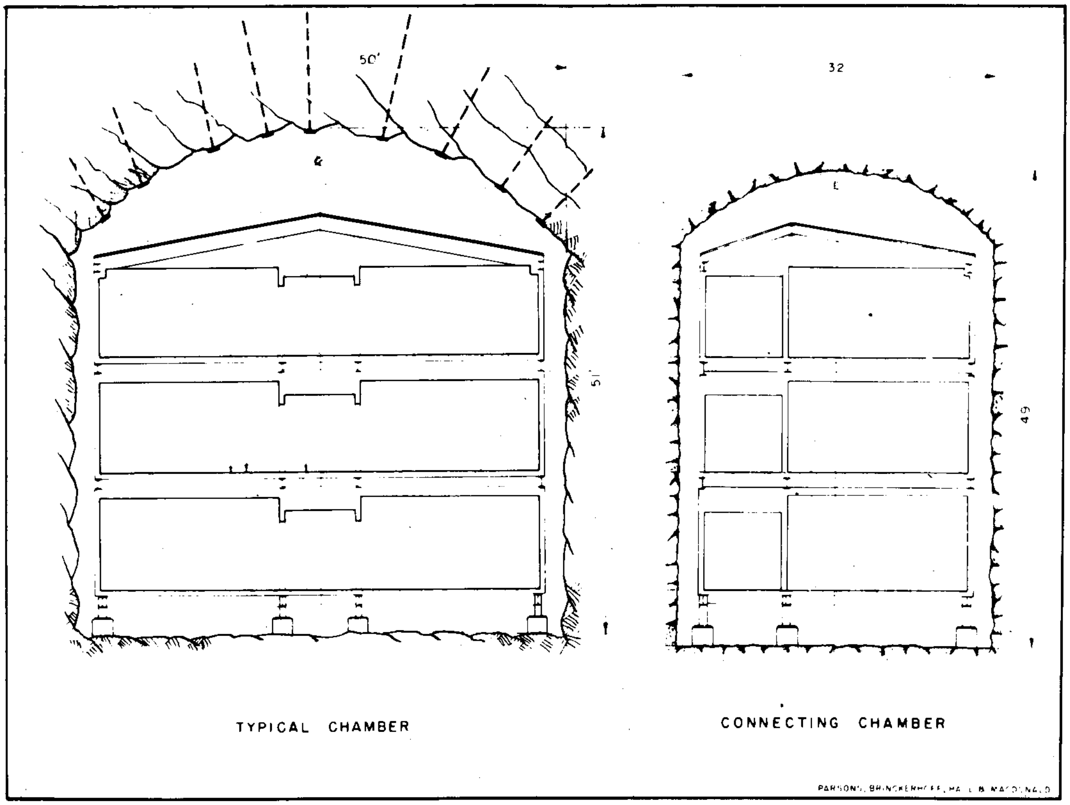
Fig. 2

Fig. 3

Fig. 4

Fig. 5

Fig. 6



TYPICAL SECTIONS - MULTIPLE STORY BUILDING

Fig. 7

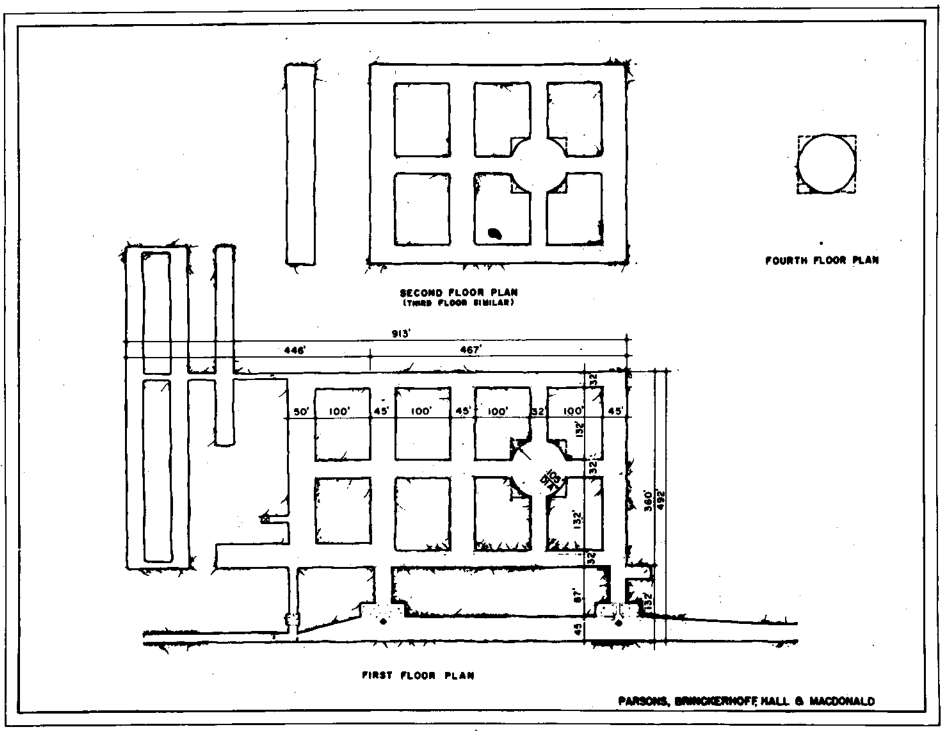


Fig. 8

GEOLOGICAL FACTORS IN CHOOSING UNDERGROUND SITES

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INTRODUCTION

The depths we are contemplating in construction of underground facilities scarcely tickle the earth's crust. Problems that develop do so because our operation represents a sudden and alien impact on the crust. This intrusion can result in seemingly mysterious reactions as we dig deeper and deeper --gushes of water at hundreds of pounds-per-square-inch pressure, sudden temperature rises, explosion of slabs from tunnel walls, heaving of the floor toward the roof, cave-ins, buckling of heavy steel I-beams, and viscous flow of wall rock.

None of these represent construction impossibilities if approached with forewarning. They have been faced and surmounted for literally hundreds of years. And, we have the advantage of modern-day construction methods and recently-evolved geotechnical theories. One interesting new factor is that a single high-velocity neutron knocks atoms out of their lattice site. Some rock is structure-sensitive to this disruption and its elastic strength thus is impaired. This means locating UG (underground) facilities below such radiation effects if the surrounding rock is critical to such danger.

Except for this concern, and thanks to rock mechanics pioneers as early as 1890, and to large UG installations currently constructed throughout the world, the geotechnical theories and experience are available to solve many of our current UG problems.

GEOTECHNICAL INFLUENCES

Among the important geological influences on UG design are:

1. Geological structure. In this case we include the European "tectonic accident," a self-descriptive term that includes faults, folding, shearing, and similar disturbances to strata.

2. Seismicity or earthquake activity.

3. Water--one of the greatest of all UG troublemakers.

4. Petrology--that is, what kind of rock is it, how did it get there, and what is its condition.

5. Geomorphology--the shape of the land surface and how it was formed; this makes it possible to unravel the subsurface picture.

These few terms, in effect, cover many textbooks, so here we only outline.

Geological Structure

Geologic structure concerns the position or attitude of the various layers that constitute the earth's crust. Because forces continuously act within the earth and because of the tremendous differences in the properties of these constituents, there is practically every imaginable crustal distortion.

Folds. Anyone who has seen photos of the human brain will recall the intricate, apparently random convolutions of the brain surface. These configurations can be compared to the shape of the crustal layer where we excavate. Rock may be folded as tightly as pleats on a closed accordion, or gently as waves on a balmy day. The type of folding influences the excavation method. (Fig. 1.) In highly disturbed beds, the full-face method might produce excessive overbreakage, heavy supports, or complete collapse. This situation might require a modified Belgian method wherein the excavation starts along the arch line and progresses no further than the spring line before placement of supporting arch ribs. Where rock is stable and not too brittle but there are considerable variations in rock types and pressures, the Austrian method of starting to excavate near the invert and gradually stoping upward might be applied. In driving through bedded or schistose rock, the tunnel should be orientated in the direction providing maximum stability; that is, with the long axis at an obtuse or 90° angle to the strike of the beds (Fig. 2c). The same criterion can be used in orientating with respect to joint or fissure systems. The degree of folding also influences the stress distribution in tunnel walls.

Faults. Occasionally, crustal stresses become too great and the folding gives way in massive tears termed faults (Fig. 3). The fault surface can be smooth, folded, or gently curved. It can be broken into a series of smaller faults that, if numerous and close together, form a shear zone. The rock within can be a miner's nightmare if it is ground into clay or dust that is permeated by water--the entire mass becomes fluid and at depths of only a few hundred feet the fluid force can crush 12 x 12 in. timbers.

Faulting may bring incompetent rock into the tunnel zone. For example, a recent tunnel into solid granite suddenly encountered soft shale with gas and water. Fortunately the contractor was forewarned because of prior geological studies.

Faults often can be found by surface study. Nature constantly strives to level the earth surface; thus, major irregularities such as narrow valleys should become suspect, as highly sheared rock is easily eroded. Of course not all river valleys indicate faults, but they merit examination if over an underground site. Abrupt offsets in otherwise continuous strata are the textbook example of faults. Any highly folded region can be expected to contain numerous faults.

Joints. A close companion of faults are joints (Fig. 4). When a brittle rock such as granite is stressed, it may fracture into more or less regular patterns. This stress can develop from orogenic (mountain-making) force or from the tensile stress caused by the cooling of the molten rock. The pattern often breaks the rock into rough rectangular blocks. If mountain-making forces repeatedly act on the rock mass, the pattern may be considerably distorted. For example, V-shaped joints (Fig. 5) are not too uncommon under these conditions. Or the pattern may be completely random, in which case the fissures generally are termed "fractures."

Fissuring can control the ultimate shape of the excavation if adequate supports are not provided (Fig. 6). Secondly, movement caused by residual stress or seismic activity may take place along the fracture lines because they represent planes of weakness. However, fracturing can be used to advantage; it can provide a convenient method of shaping the excavation or act as a path of non-resistance for relieving major rock stress before the tunnel enters this stressed zone; thus major fall-outs or rock bursts do not occur.

Aseismic Design Factors

The mention of stress relief generally brings to mind questions concerning the influence of seismic activity. If feasible, it is advisable to locate the structures in a region of minimum seismicity. Although the Bikini tests scarcely affected seismographs, major earthquake shocks often jump the needles off the chart. Thus, it is easily inferred that the forces involved in earthquakes are too tremendous to be resisted by man's puny structures. If a deep excavation were located astride a fault in a seismically active area, a tremor might move rock in opposite directions along the fault. This actually occurred in a railroad tunnel in California where the rails were driven into the tunnel walls (Fig. 7). If the underground chambers can be located between fault zones, serious destruction might be avoided, although a severe shock might create a new fault zone.

Residual Stress

A current problem is the relation between seismic activity and residual stresses as evidenced by rock burst and bump phenomena. Does the quake induce unrelieved strains in the crust that ultimately produce rock bursts in a mine? To date, despite intensive study, the answer is negative. No relationship has been shown between occurrence, or even frequency, of rock bursts or bumps and earthquakes. Yet, it is generally agreed that the bursts are caused by stored stresses in the rock. They occur without warning unless microseismic detection gear is used--and this has been ineffective in predicting bumps. Current experiments in South Africa indicate a possibility of at least reducing frequency of bumps by drilling 10-ft long holes at 5-ft centers into the working face. Light charges are exploded to shatter the rock and thus reduce stresses near the face to zero. These stress phenomena lead directly to the problem of residual stress and resulting stress relief.

When the earth crust is contorted without failing by faulting or fracturing, stresses produced by the tectonic forces have to be gradually dissipated, and thus a quasi-storage of tectonic energy occurs in the surrounding rock. When man excavates into this stressed mass, the rock suddenly finds the opening a convenient place to go, and forthwith goes! Frequent relief by bumps or sudden popping of slabs from walls occurs at depths below 500 ft, and the frequency increases with increasing depth. Such strain relief also is seen in deep canyon walls (Fig. 8) or quarry excavations.

In large underground excavations, residual stress can be a major problem owing to the magnitude of the force. In Galt Mine in Canada, tensile stresses up to 3000 psi were measured. In Australian underground powerhouse construction horizontal pressures in excess of vertical ones resulted in cracking of massive concrete ribs supporting the roof, with the crack pattern at the arch haunch and top of the crown demonstrating tensile forces.

Detrimental stress relief sometimes can be avoided (Fig. 9). UG excavations should be situated as far as possible from highly disturbed strata, particularly tight folds, intrusive zones, or feathered fractures, which generally indicate concentrations of unrelieved stress in or near them. High stress concentrations also can be caused by improper pillar spacing, too rapid an excavation with too slow a placement of supports, tunnel sections with relatively square corners (Fig. 10), placing small openings near large ones, or introduction of sudden temperature changes within the chamber that, because of different coefficients of expansion in mineral constituents, result in the stressing of the rock.

Measuring Devices. The problem of proper design to avoid these problems is difficult because of the lack of suitable tools to measure residual stress and accurate theories as to its origin. Efforts are being made to obtain quantitative measurements of residual stress and its lateral extent from the tunnel wall. One method used to date consists of SR-4 strain gages cemented to excavation walls, relieving the rock by boring a core from around the gages and then recording the resulting strains (Fig. 11c). This method has proved sufficiently accurate to permit the construction of pressure tunnel linings with a minimum of re-steel in two different locations in the United States.

Also deserving special attention is the use of geophysical methods. Recent seismic tests in the 20-ft diameter Clear Creek Tunnel (U. S. Bureau of Reclamation) indicated zones of variable rock strength extending outward from the tunnel periphery, although there was no coincident change in rock type. There were four definite velocity layers that ranged from 1700 ft/sec within 3 ft of the wall, 3950 ft/sec 3 to 10 ft away, to 18,300 ft/sec 18 ft away. These layers are within the stressed zone created by the excavation (Fig. 12). The lower values probably represent the Trompeter Zone--the area of rock actually fractured by excavation stress--whereas the higher values are in a zone of unfractured but highly stressed rock. It appears that rock undisturbed by the excavation was not encountered or, more possibly, was masked by the peak velocity layer at 18 ft. The total influence of the excavation can vary in extent depending upon tunnel diameter, depth below the surface, intensity of residual stress, and proximity to tectonic accidents. For example, at depths of 1200 to 2000 ft, the Russians found this influence to extend 325 ft beyond the working face.

Influence of Rock Type. A current defect in tunnel stress analysis is that it does not incorporate influence of rock type on the stress distribution (Fig. 13). Admittedly this failure is partially attributable to lack of knowledge, but its possible effects certainly should not be ignored. Statistical analysis of some 200 tunnels has shown a slight relation between percentage of overbreak (a value that is believed to provide a qualitative measure of rock strength and stress) and tunnel diameter or size (Fig. 14). This same study is inconclusive as to rock type vs overbreak primarily because of the lack of petrographic data for the tunnels analyzed. Despite these meager results, experience certainly has shown that a slight idea of the type of rock to be encountered is desirable before proceeding with tunnel operation.

Underground construction experience shows that the support type definitely may depend upon the rock type; a dense hard granite usually provides a far cleaner section with a minimum of supports than does a soft, platy shale. In the latter case, shale can be indicative of high-stored stresses that would slowly push down columns and walls in the chamber. Dr. Duvall has described the influence of rock type on the effectiveness of rock bolts. Columnar basalts can create a hazardous tunnel because of their complex jointing and often severe fracturing--not to mention their potential as an aquifer. Chalk and soft calcareous rocks are not often thought of as desirable UG sites but the Germans successfully placed entire manufacturing plants in huge chalk chambers prior to World War II.

Elastico-Plastic Effects

It is a whim of nature and a frustration to the tunnel designer that there are such diverse reactions. The clue to these reactions occurs in recent rock mechanics research. Rocks with high plasticity can be expected to react like chalk when excavated. Brittle rocks, those with high elasticity but low plasticity, would react like basalt. The theories to explain these reactions will have to be developed by rock mechanics research. Many theorists start with a one-line assumption that the rock mass is isotropic, homogeneous, and elastic, and they further ignore nature by considering a semi-infinite mass. They then proceed to extend their theories to decimal-point accuracy. This approach is so remote from reality that the modern rock mechanics researcher goes back to the one-line assumption and finds that he has not only an aeolotropic material, but one that can be elastic, plastic, granular, or liquid!

Until rock actually fractures under stress it should be analyzed as an elastico-plastic material; that is, when the imposed load exceeds the elastic strength, the specimen will react plastically until ultimate failure occurs. Under triaxial compression, marble appears to flow and actually increase in volume; high pressures at great depths may cause brittle rocks like granite or basalt to react plastically and even flow. A miniature analogy occurs in laboratory tests: When the test load reaches a value where the stress-strain relation is no longer linear but still below the ultimate failure point, and the load is suddenly released, the rock does not return to its original size. The amount of this hysteresis or set decreases with repeated applications and releases of loads. Thus, it is found that these repeated load applications below the ultimate failure value increase the elastic strength. This seems to indicate that the work-hardening of metals also can occur in rock.

Once the rock does fracture, however, it can theoretically be assumed to respond like granular material. To oversimplify, this fractured mass might be regarded as analogous to a sand pile and analyzed by soil mechanics theories that relate cohesive strength and internal friction to the overall shear strength. This method of analysis applied to rock still is embryonic, due to insufficient research. However, the application of this analytical method will require determination of the internal friction developed between fragments. That is, the influence of rough surfaces of the individual fragments plus their partial interlocking will be considerably greater than analogous influences that occur on a microscopic basis in soil aggregates.

Rock Type vs Excavation Methods

Rock type also has a considerable influence on the cost of excavation methods. Soft rocks such as shale and tuff are most efficiently blasted with low-cost, low-velocity explosives like the nitramons; the hard, brittle rocks are efficiently shattered by high-cost, high-velocity explosives such as 40 or 60 per cent dynamite.

Studies also indicate direct correlations between rock type and drillability--that is, the type of blast-hole rig used and the speed and bit wear in such rigs (Fig. 15). Drillability is affected primarily by crystalline structure and the nature of the fracturing. Hard rock composed of tightly interlocked angular crystals will drill slower and cause higher bit wear than rock with rounded crystal contacts, weak cementing material, or altered constituents. A rock containing both hard and fibrous minerals is one of the most difficult to drill. Extremely fractured rock or dense granites or limestones usually are most efficiently drilled by percussion rather than by rotary rigs. The latter would be satisfactory in shales or similarly soft but cohesive rocks. Rock containing minerals with vastly different coefficients of expansion can be effectively penetrated with jet-piercing rigs that burn oxygen and petroleum base fuel. These comments illustrate a few uses for microscopic study of rock prior to construction operations.

Petrofabrics

Recently attempt has been made to use petrofabrics or rock texture to estimate strength properties (Fig. 16). This method has progressed sufficiently to prove that it can minimize the need for extensive laboratory load tests. The microscope will disclose the nature of contacts between minerals or grains and their orientation, the type and distribution of cementing material, strain lines or inherent fractures, and amount of alteration.

Of major importance are the contacts and their orientation (Fig. 17). A rough parallel orientation in the minerals can be extrapolated to mean slabbing tunnel walls and excessive shattering when blasted. Conversely, such rock may not be explosively influenced by residual stresses, because the orientated contacts represent planes of weakness that will permit gradual stress relief by slippage along these planes (a seemingly plastic reaction). Interlocking contacts may indicate rock difficult to shoot but having easily-controlled blasting effects with minimum of overbreak (Fig. 18). Also they are indicative of high strength and thus minimum support. On the debit side, however, this interlock prevents the gradual relief of residual stresses; thus, explosive rock bursts can occur.

Strain patterns in crystals may indicate residual stresses, but to date no way has been found to use this as a quantitative measure of the forces (Fig. 19). It can be postulated that the tectonic forces creating the strain were applied normal to the long axis of the patterns; thus, there might be less stress relief effect if the tunnel axis is orientated normal to the pattern. Fractured, rounded contacts, or poorly cemented crystals may cause excessive shattering under blasting and little or no resistance to high residual stresses (Fig. 20).

Weathering

Another phase of petrologic study is the effect of weathering (Fig. 21), which has been found over 650 ft below the ground surface. In deep tunnels, weathered rock that is saturated or under high lateral pressure may flow into the tunnel or require steel liner plate supports. Since such material would react plastically or like a soil, it could be expected to interfere with tunnel stress analysis, based on elastic theory. Hard granite can completely deteriorate to a cohesionless mass of sand or sticky clay and may be overlooked since it can superficially resemble the unaltered rock. An example of this was recently found in a tunnel about 450 ft underground. The wall rock appeared to be good granite, but when struck with a geology hammer the point was embedded almost one inch in clayey material. There was no surface evidence of the existence of this altered zone.

Other weathering effects can occur when a rock stratum is exposed during its geologic history. The surface will weather and erode; additional beds will be deposited and deeply bury this eroded surface. When encountered in tunnels, such unconformities warn of different rock types above and below the old weathered surface, alteration of rock adjacent to it, and the possibility of ancient buried channels that contain highly pervious material and possibly water.

Weathering or alteration of the contact between sound bedrock and fault zones or intrusives (Fig. 22) may create the need for heavy support. This alteration can result from the high temperatures accompanying the intrusion, from heat generated by the faulting stresses, or because the contact provided a seepage path for groundwater or deep-seated water.

Air slaking of rock when freshly exposed is another weathering phenomenon. This continual flaking is common in moist, soft, or clayey shales or silt-stones, but occasionally occurs in altered granite or schists. The deterioration can be caused by the rapid evaporative effects of surrounding air currents or by a reverse process wherein excessive moisture is forced between constituent minerals by capillary or ionic action. Unless controlled by protective coatings (Fig. 23), deterioration can progress to considerable depth. And, protection is vital when concrete is to be bonded to the rock surface.

Water

Most tunnel contractors will agree that water (Fig. 24) is one of their biggest headaches. It also is to the engineering geologist; only an innocent new graduate would dare to offer a quantitative estimate of flows to be encountered. Qualitative estimates can often be given, however, such as: "The flow will not affect construction;" or "Large volumes may completely shut down the job so protective measures should be taken." The general criteria presented here contain the proverbial salt grains, as their absence or presence seldom are completely indicative of potential subsurface conditions.

Generally, it is advisable to locate the excavation above the groundwater table or intersect it where good drainage can occur. Thus, water trouble would result only from moderate surface seepage or the occasional and unpredictable tapping of a deep-seated spring. Dried-up spring deposits at the surface may indicate active flows at tunnel grade. Thermally active areas such as Paricutín in Mexico would indicate deep-seated cold or hot water or even steam. Permafrost zones often carry unfrozen water courses. Surface valleys may mean faults and faults may mean water trouble at tunnel level. Locating a tunnel, without adequate investigations, beneath surface bodies of water can be a losing gamble, as was sadly illustrated when the Moffat Tunnel was driven through the Continental Divide in 1930. The bore was about 2000 ft below one of many lakes near the top of the Divide; there is no fishing in this lake now because a wide shear zone acted as a conduit between the lake and the tunnel.

Extensive fractures may indicate water problems if there is a source of supply. Igneous rock often provides good water wells because of extensive underflows through fissures. Cavernous limestones and basalts often carry actual flowing streams (Fig. 25). Tunneling through permeable sedimentary rocks may be somewhat wet if there is a source of supply.

Sometimes a tunnel can be safely located below a perched water table because an impervious strata seals off the underlying tunnel (Fig. 26). In this case, or where the table is held below tunnel grade by an impervious strata, there exists the possibility of a break-out into the tunnel owing to hydrostatic pressure.

Rock Temperature

Excavation at considerable depth also should be preceded by study of expected geothermal gradients (Fig. 27). The rock temperature roughly increases at the rate of 1° Centigrade per 100 ft of depth below the surface. This gradient is inexact because it is influenced by surface relief, air and ground temperatures, difference in thermal conductivity of various rocks, and the dip of the strata because conductivity of rock is greater parallel to the bedding. The vertical strata in Gotthard Tunnel resulted in a gradient of 154 ft whereas the flat-lying beds intersected by the Simplon Tunnel resulted in a 121-ft gradient. Fissured or highly porous rocks tend to decrease the temperature gradients because of their sorptive properties, particularly if cool surface water penetrates the fissures. Under mountain peaks, rock temperature increases at a much slower rate than under valleys.

Temperature conditions are not to be ignored because in deep tunnels more costly air-conditioning facilities may be required, not to mention discomfort during construction. At Tecolote Tunnel in California, 160° heat resulted in the miners being transported to the face in mine cars filled with cold water.

Permafrost

A converse temperature problem is met in Arctic-type terrain as a result of permafrost, a zone where subsurface temperatures are at or below freezing for a period of at least 2 years. Permafrost zones are known that have a vertical extent in excess of 2000 ft. And, because of extensive glaciation normal to such terrain, overburden soil may be hundreds of feet deep. The top of the zone, or permafrost table, can be at or near the ground surface and may show a peculiar polygonal pattern (Fig. 28). Sad experience in surface construction in the Arctic indicates possible tunnel problems. The mere presence of a surface building without heat can raise ground temperatures sufficiently to thaw permafrost. If the thawed material is relatively impervious soil, it may be over 200 per cent saturated and the building slowly sinks from view! Notice the gaps below the walls and tilted frames in Fig. 29. Permafrosted rock or dry or pervious soil probably would not be troublesome, other than its influence on temperature control in the UG site. However, if ice is the cementing agent in a fractured or fragmented rock, excavation collapse can be expected when the permafrost thaws; frozen water-courses in fissures are not unusual and would be trouble-makers if thawed by the construction or operation conditions in the tunnel.

The most successful method of combatting permafrost problems is the passive one wherein the disturbance to the permafrost regime is held to an absolute minimum. Heavy insulation or even refrigeration may be required if operations involve above-freezing temperatures. One of the most serious problems in permafrost-area work is maintenance of portal cuts. When the portal area is stripped of vegetation to start excavation, the permafrost thaws because the natural insulation is removed. Thawed permafrost soil literally presents a sticky construction problem--the ground actually flows and heavy equipment will bog down or completely disappear!

Incident to this problem is the construction of UG sites in snow and ice fields. The military work in Greenland and Antarctica indicates the feasibility of this, but constant maintenance is necessary because of the constant creep and subsequent closure of snow tunnels.

Portal Geology

A necessary adjunct to UG site analysis is a geotechnical study of access or portal areas. A major factor is the possibility of landslides (Fig. 30) or snowslides and rock falls. These can cause construction delays or later can result in blocking of access to the permanent installation. It is not feasible here to discuss stability analysis methods for open slopes cut from rock or soil. However, in our situation, the analysis should introduce loading factors that will compensate for the concussion and vibration forces of nuclear blasts.

Rock falls can be caused by excavation operations or seasonal freezing and thawing. The stability of any open slope will be affected by seismic shocks. Another problem peculiar to our study is that the extremely high destructive force of a nuclear blast could disturb surface topography sufficiently to divert surface water courses into the access areas of the UG sites.

INVESTIGATION METHODS

Because the technical literature has excellent coverage of geologic subsurface investigation methods, only a few pertinent ones are discussed here. Rotary core drilling is one of the most efficient methods and its efficacy is directly proportional to the diameter of the hole (Fig. 31). The NX (2-1/8 in. core) is the minimum size desirable, whereas the best are man-sized shafts as large as 6 ft in diameter drilled by special rigs (Fig. 32) or exploratory tunnels. One of the most important purposes of these holes is to evaluate joint and fracture systems. Large shafts or tunnels can be used also as a base for drilling holes at various angles for complete exploration of the UG site. Whether the core is 2-1/8 in. or greater diameter, it can be used for laboratory tests; also the core hole itself is valuable because, as discussed in Mr. Nesbitt's paper, a camera can be lowered into it to take orientated photographs of in-situ jointing conditions. A modification of the TV camera suitable for examining the interior of bore holes is also available.

There are indirect subsurface investigation methods such as geophysics. Seismic or resistivity equipment can locate faults, determine rock types, permeabilities, depth of weathering, and bedrock surface contours. The resistivity method has been successfully used to locate deep fracture zones (Fig. 33). The seismic method has been used to measure elastic moduli in situ, as there is an empirical relation between velocity of seismic wave propagation and the density and elasticity of the rock (Fig. 34). This application still is so new that considerable correlation is needed with other types of field and laboratory tests. The major correlation difficulties arise because the geophysical test is a dynamic method, whereas the laboratory load tests are static (Fig. 35). As a matter of interest, at present no two test methods appear to correlate. The major problem is to determine which method provides the answer most applicable to the proposed design, and if the results are subject to interpretation by empirical formulas based on the theory of elasticity.

A point in this regard is the use of laboratory triaxial shear tests (Fig. 36). In these, the rock specimen can be placed under simultaneous axial and lateral stress and either load can be varied at will. It provides a laboratory simulation of in-situ lateral pressures at great depth. The bogey factor is that the loads caused by residual stresses that are known to occur at depth cannot be added. They cannot because, as yet, efficient measuring instruments have not been developed. It is found, however, that even if the effects of residual stress are disregarded, the analysis of 120 triaxial tests shows that the lateral strain rapidly increases in proportion to axial or vertical stress and exceeds it at depths within our realm of consideration.

CONCLUSION

The final and most important factor in investigations is to present the results in a manner understandable to those not holding higher degrees in geology. It seldom is desirable to use decimal-point accuracy because the only homogeneous thing about nature is heterogeneity. Basically, the problem is to accumulate adequate and correct geotechnical data by the combined efforts of engineers and geologists. UG site problems, in reality, involve only two relatively new concepts--unusual disturbance of natural conditions owing to the extremely high forces developed by nuclear weapons, and the use of a science still in labor pains but of inestimable value when born--rock mechanics. This birth will permit us to, one, minimize the disturbance to natural underground conditions, and two, find out what these conditions are before we start!

DISCUSSION

MR. LARS DE JOUNGE (Sandvik Steel, Inc.): You mentioned a tunnel 6 ft in diameter and 1,200 ft in depth. Where was that?

MR. JUDD: This is not a tunnel but a shaft. It is in South Africa. It is a ventilation shaft for one of the mines. A similar one has been reported in Brazil.

FIGURES

23. Protective coating to retard weathering.

24. Underground water flow in relation to tunnels. Principles of Engineering Geology and Geotechnics.

25. Effects of underground flowing streams.

26. Possible tunnel flooding due to hydrostatic pressure. Principles of Engineering Geology and Geotechnics.

27. Diagram of geothermal gradients. Principles of Engineering Geology and Geotechnics.

28. Polygonal pattern of permafrost at surface.

29. Construction damage to building on permafrost.

30. Landslide.

31. Exploratory rotary core drilling.

32. Shaft-sinking machine. "Sinking Large Diameter Mine Shafts by Rotary Drilling," Victor Zeni and T. N. Williamson, Mining Engineering, published by American Institute of Mining, Metallurgical, and Petroleum Engineers, Inc.

33. Method of delineating fractured zones in bedrock. "Subsurface Investigations of a Plant Site," L. Scharon, R. Uhley, and Tavi Meidav, Mining Engineering.

34. Seismic modulus determinations. Proceedings, XXth International Geological Congress.

35. Field seismic tests compared with laboratory values.

36. Laboratory triaxial shear tests.

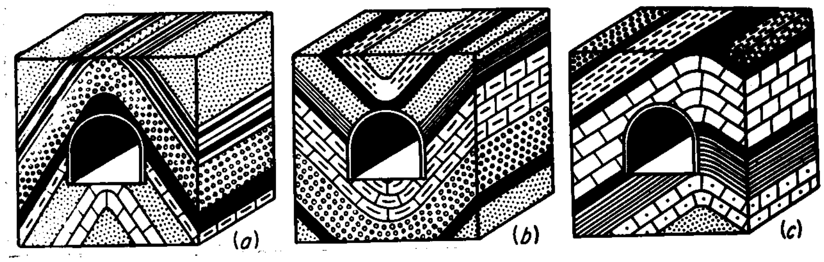


Fig. 1

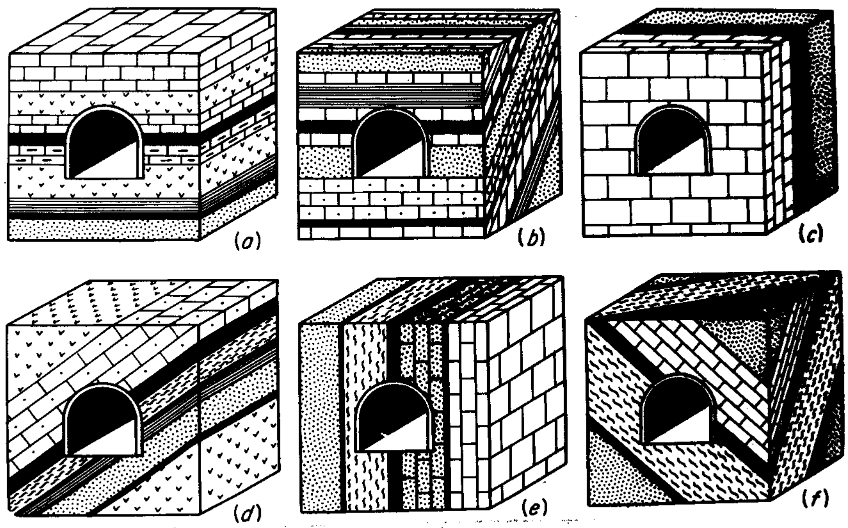


Fig. 2

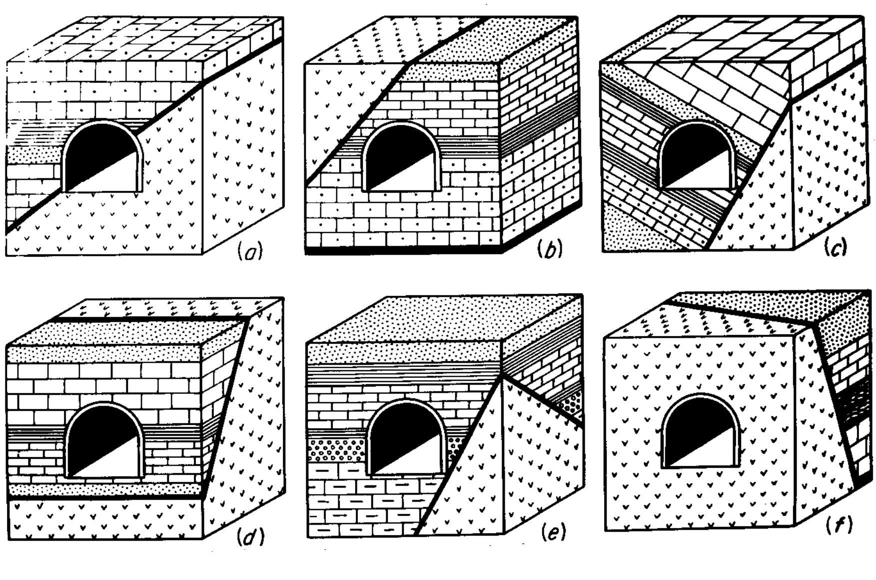


Fig. 3

Fig. 4

Fig. 5

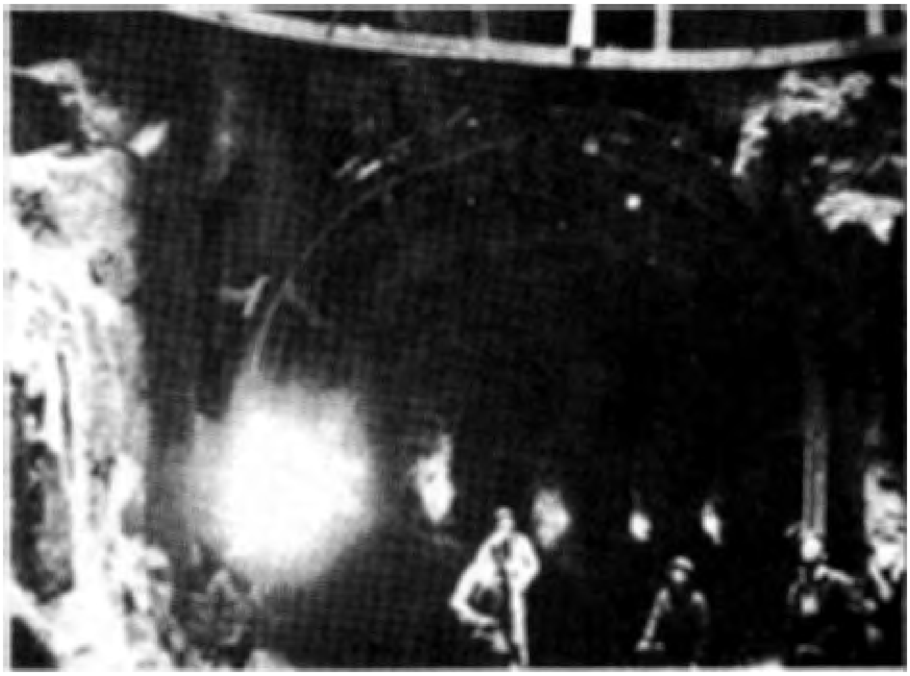


Fig. 6

Fig. 7

Fig. 8

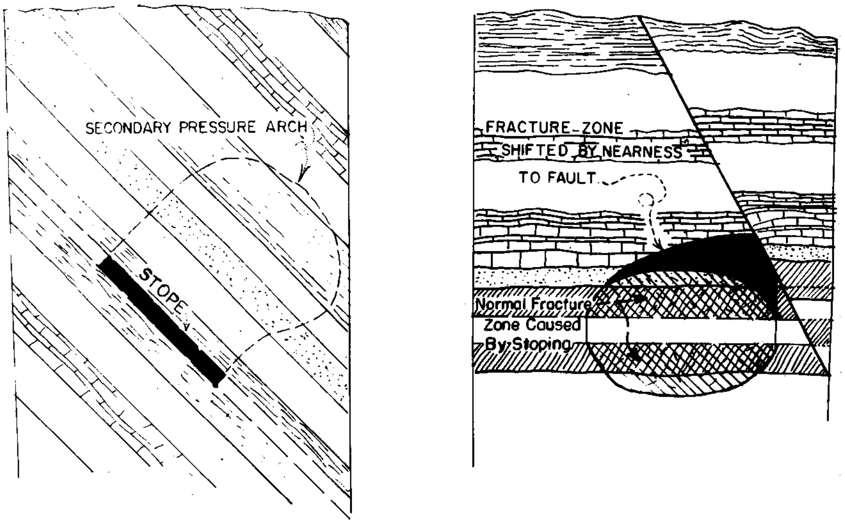
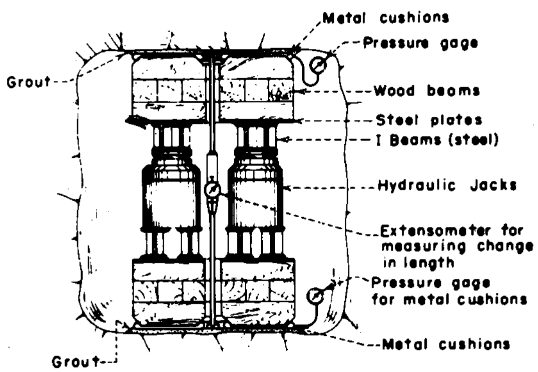
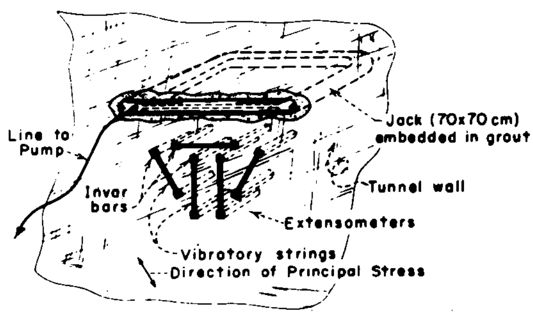

SECONDARY PRESSURE ARCH
STOPE
FRACTURE-ZONE
SHIFTED BY NEARNESS
TO FAULT
Normal Fracture
Zone Caused
By Stoping

Fig. 9

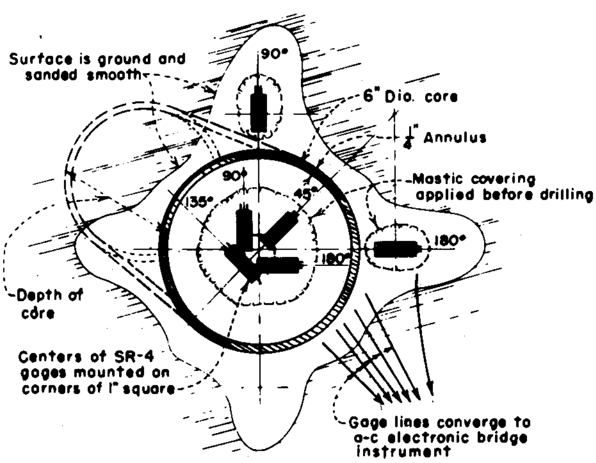
Fig. 10



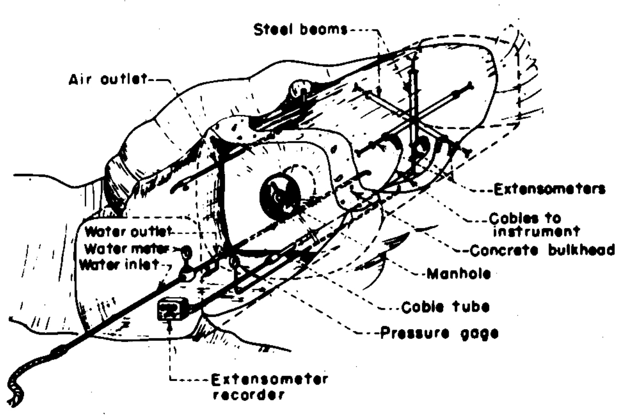
(a) JACK LOADING TEST



(b) FREYSSINET TEST



(c) STRESS RELIEF TEST



(d) WATER LOADING TEST

FIELD LOADING TESTS

Fig. 11

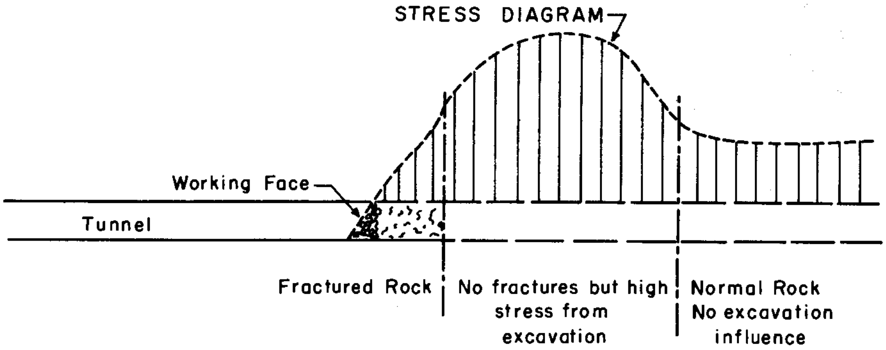
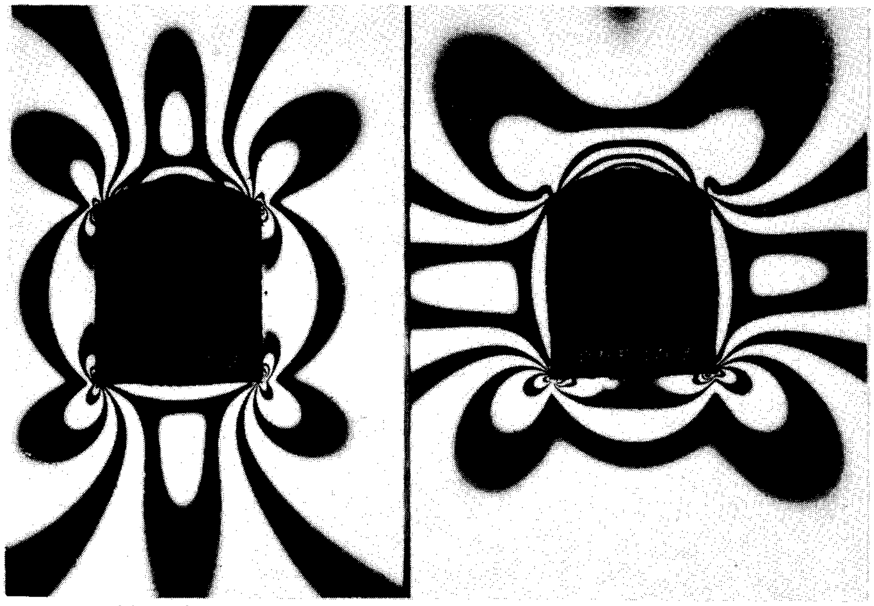


Fig. 12



Vertical load

Horizontol load

Fig. 13

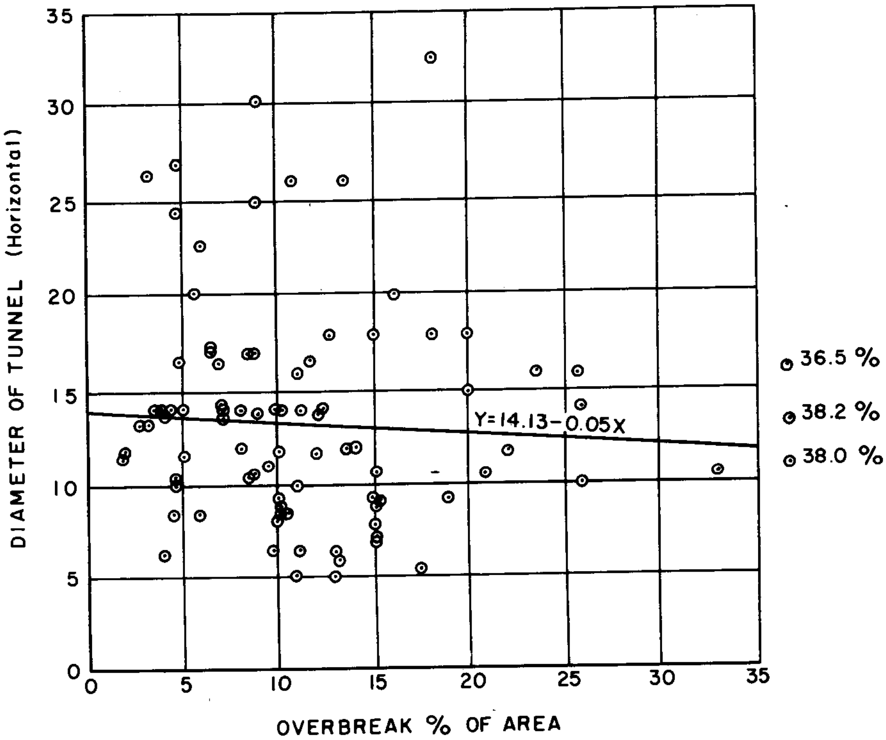

DIAMETER OF TUNNEL (Horizontal)
OVERBREAK % OF AREA
Y=14.13-0.05X
36.5 %
38.2 %
38.0 %

Fig. 14

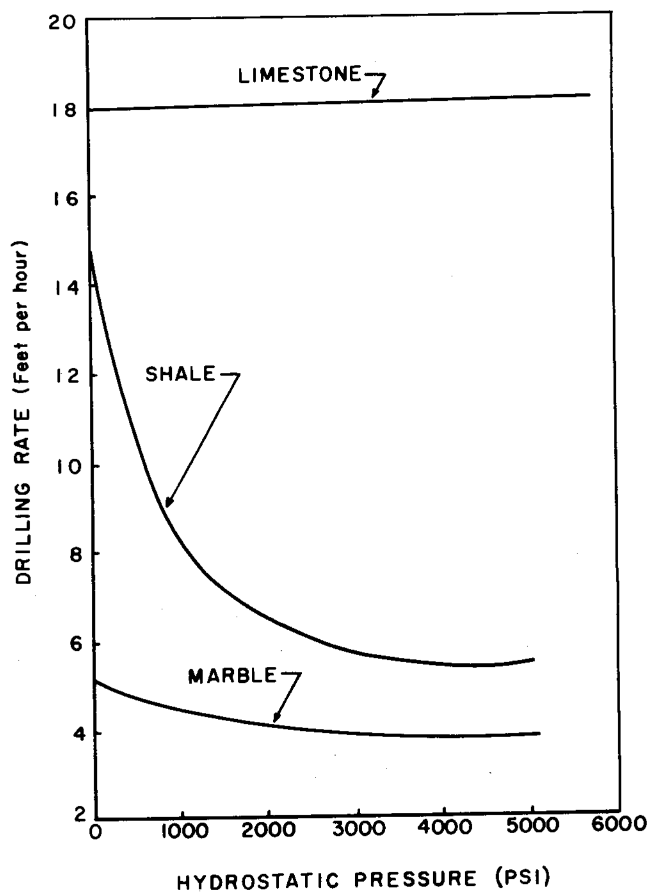

DRILLING RATE (Feet per hour)
HYDROSTATIC PRESSURE (PSI)
LIMESTONE
SHALE
MARBLE

Fig. 15

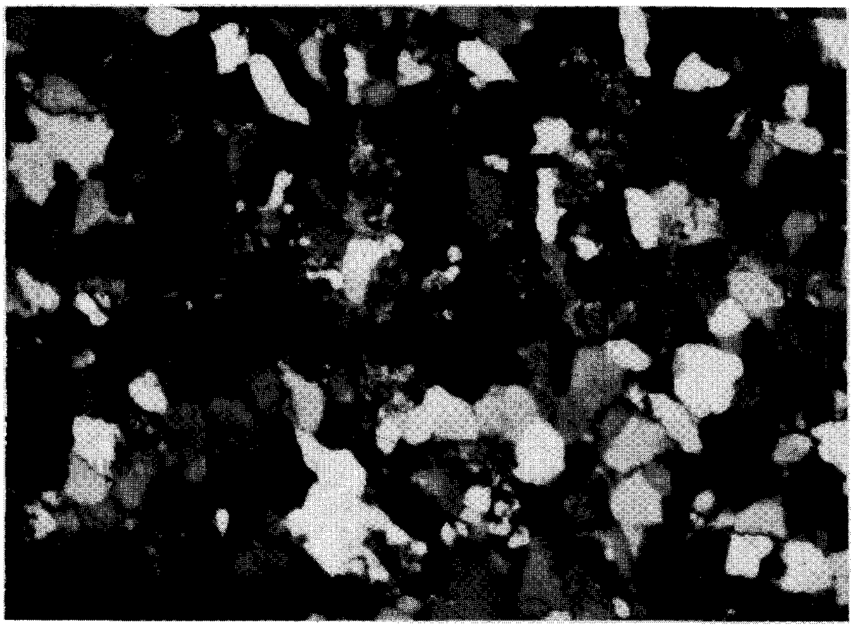


Fig. 16

Fig. 17

Fig. 18

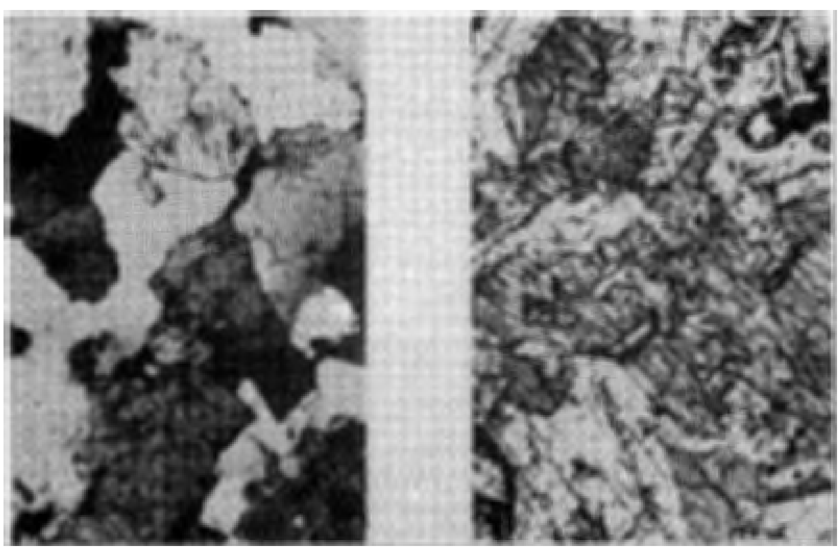


Fig. 19

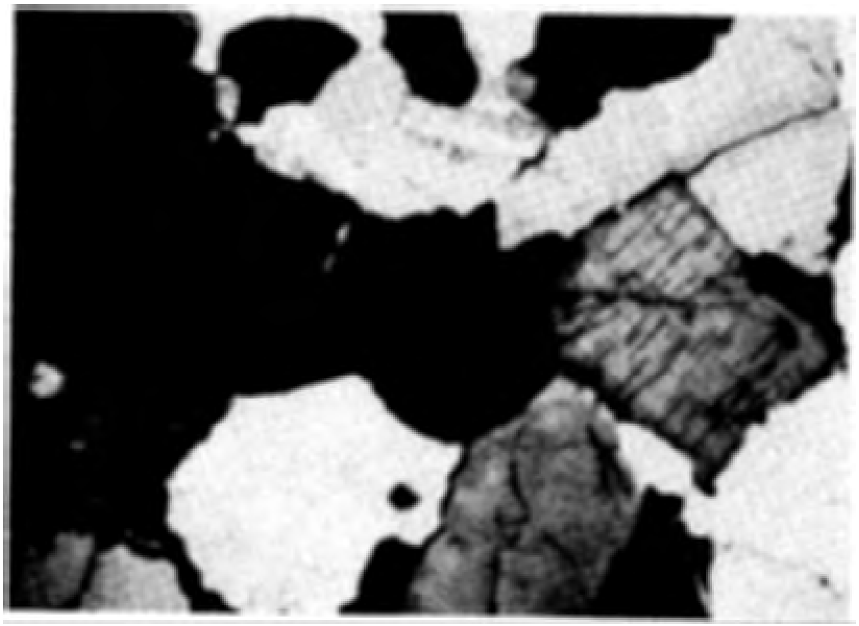


Fig. 20

Fig. 21

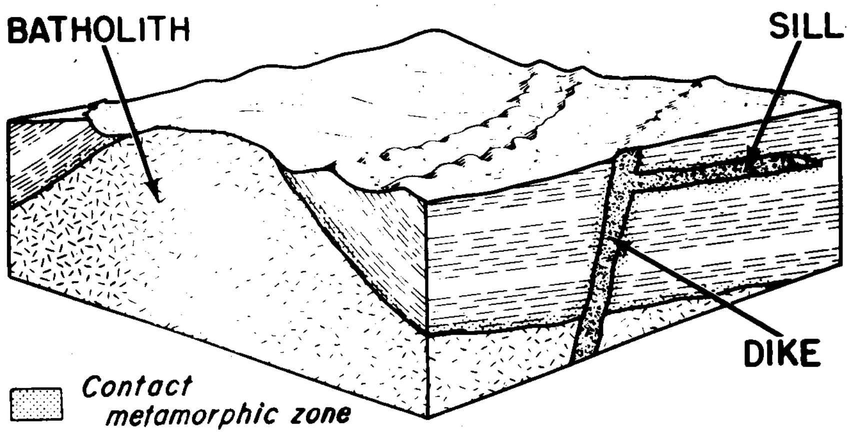


Fig. 22

Fig. 23

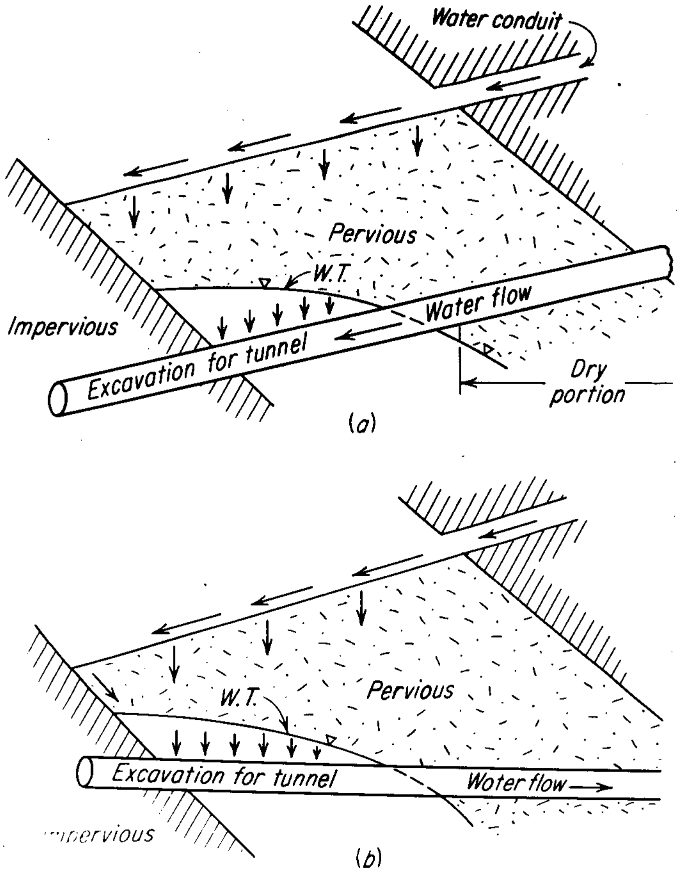

Water conduit
Pervious
W.T.
Water flow
Impervious
Excavation for tunnel
Dry portion
(a)
W.T.
Pervious
Excavation for tunnel
Water flow
Impervious
(b)

Fig. 24

Fig. 25

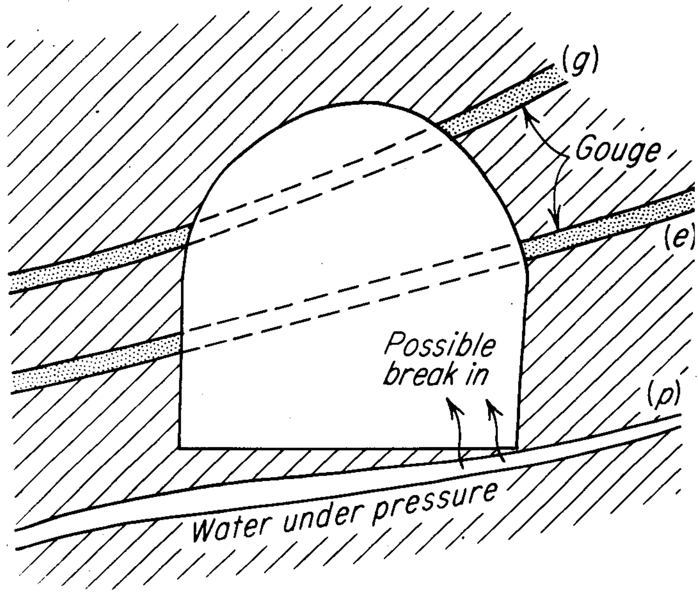

(g)
Gouge
(e)
Possible
break in
(p)
Water under pressure

Fig. 26

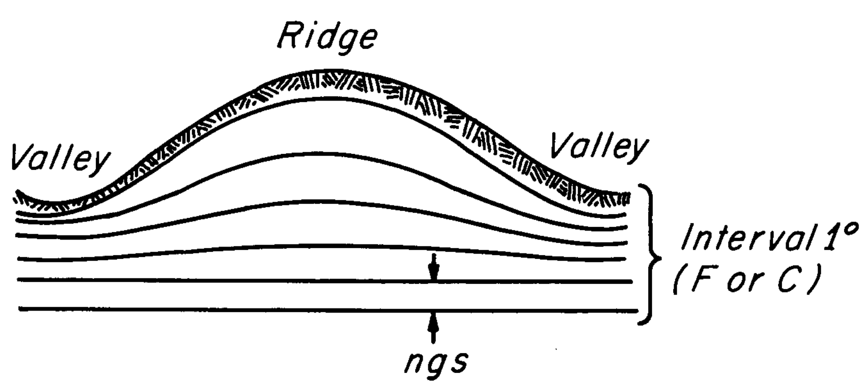

Ridge
Valley
Valley
Interval 1°
(F or C)
ngs

Fig. 27

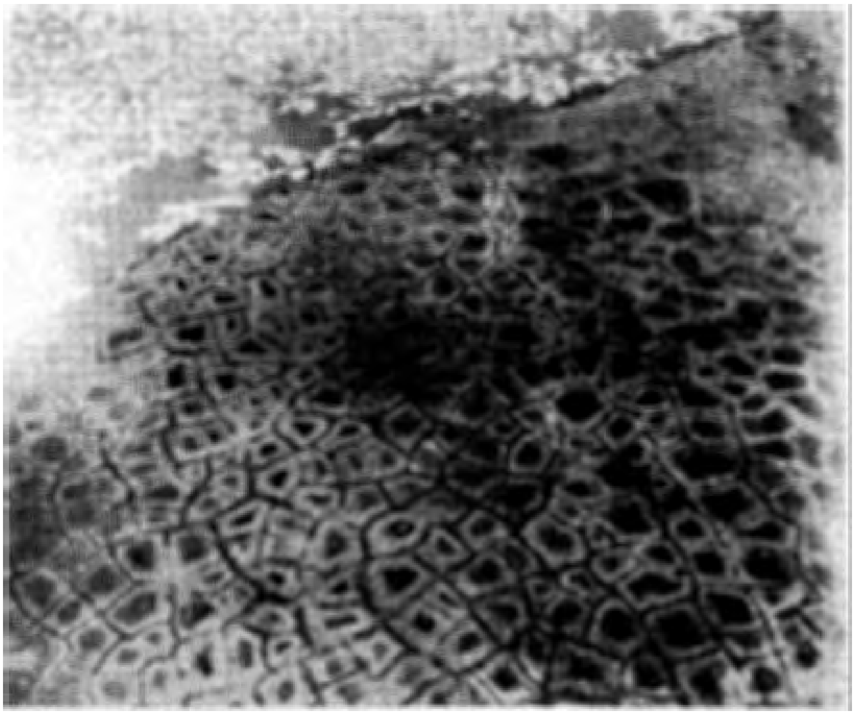


Fig. 28

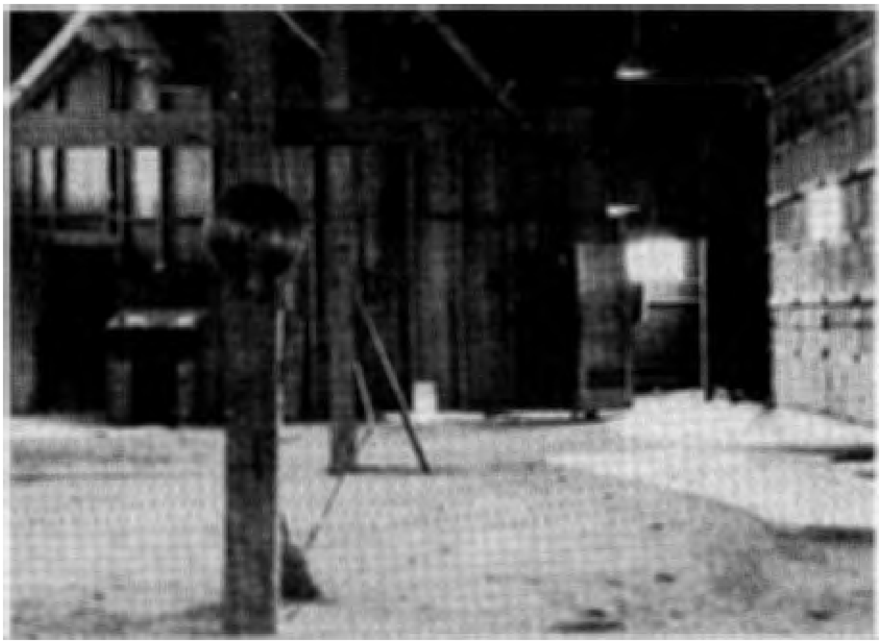


Fig. 29

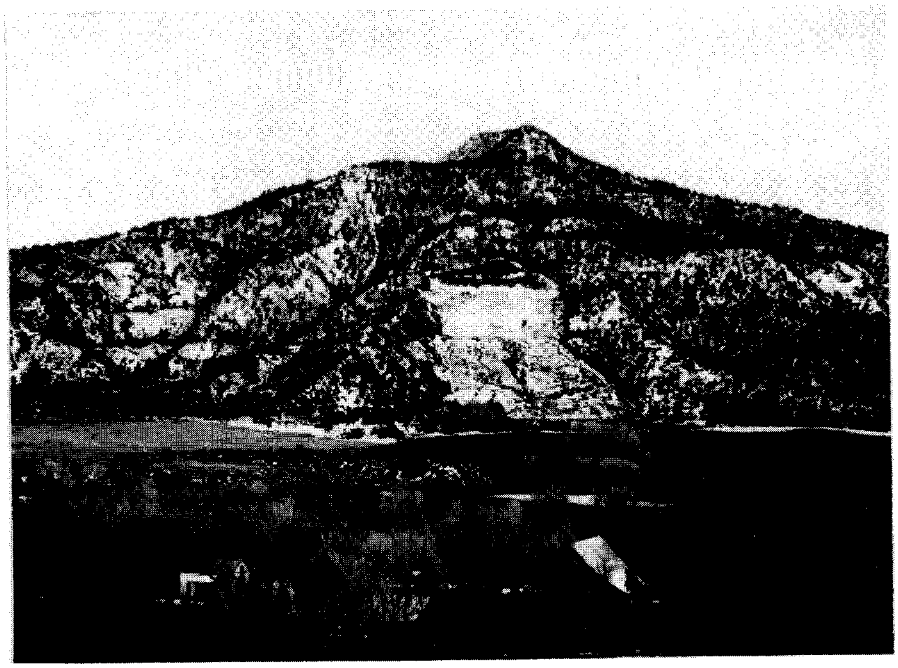


Fig. 30

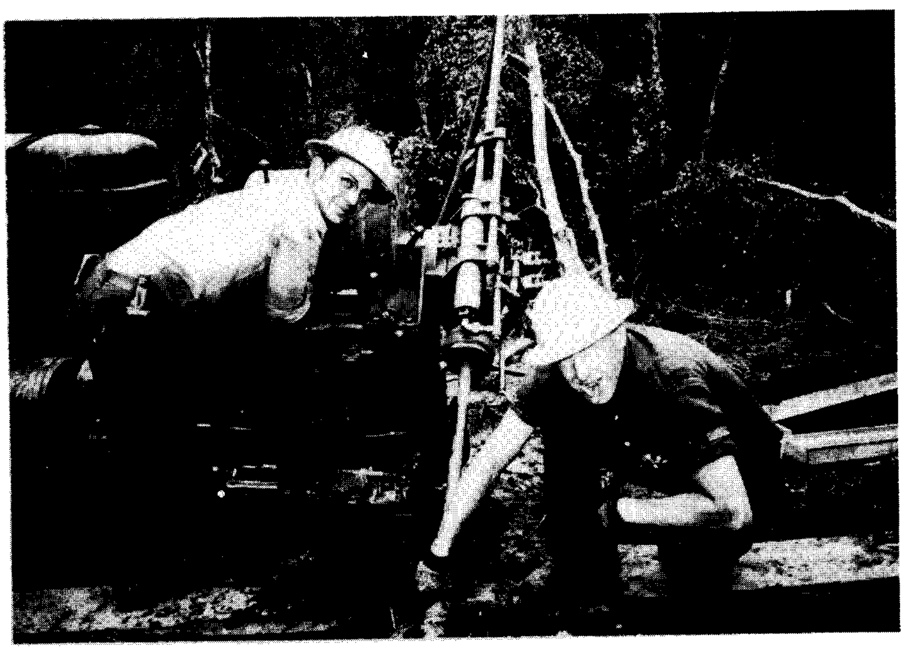
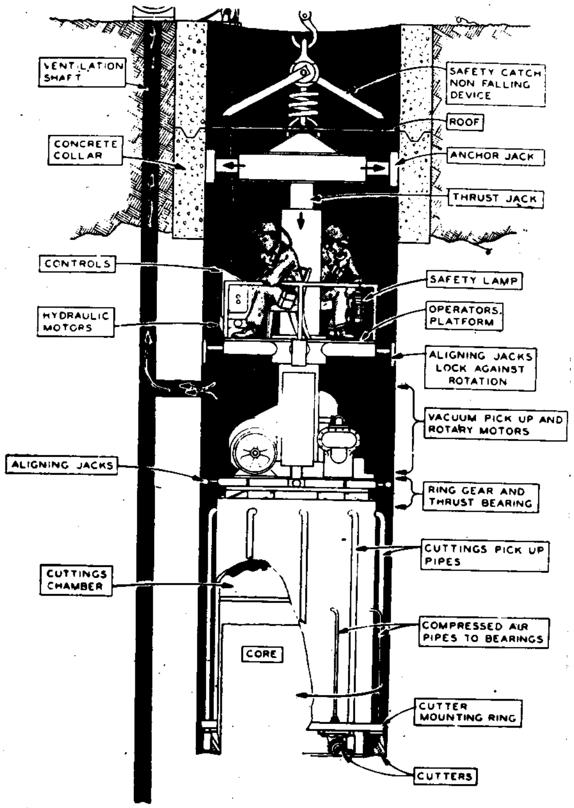
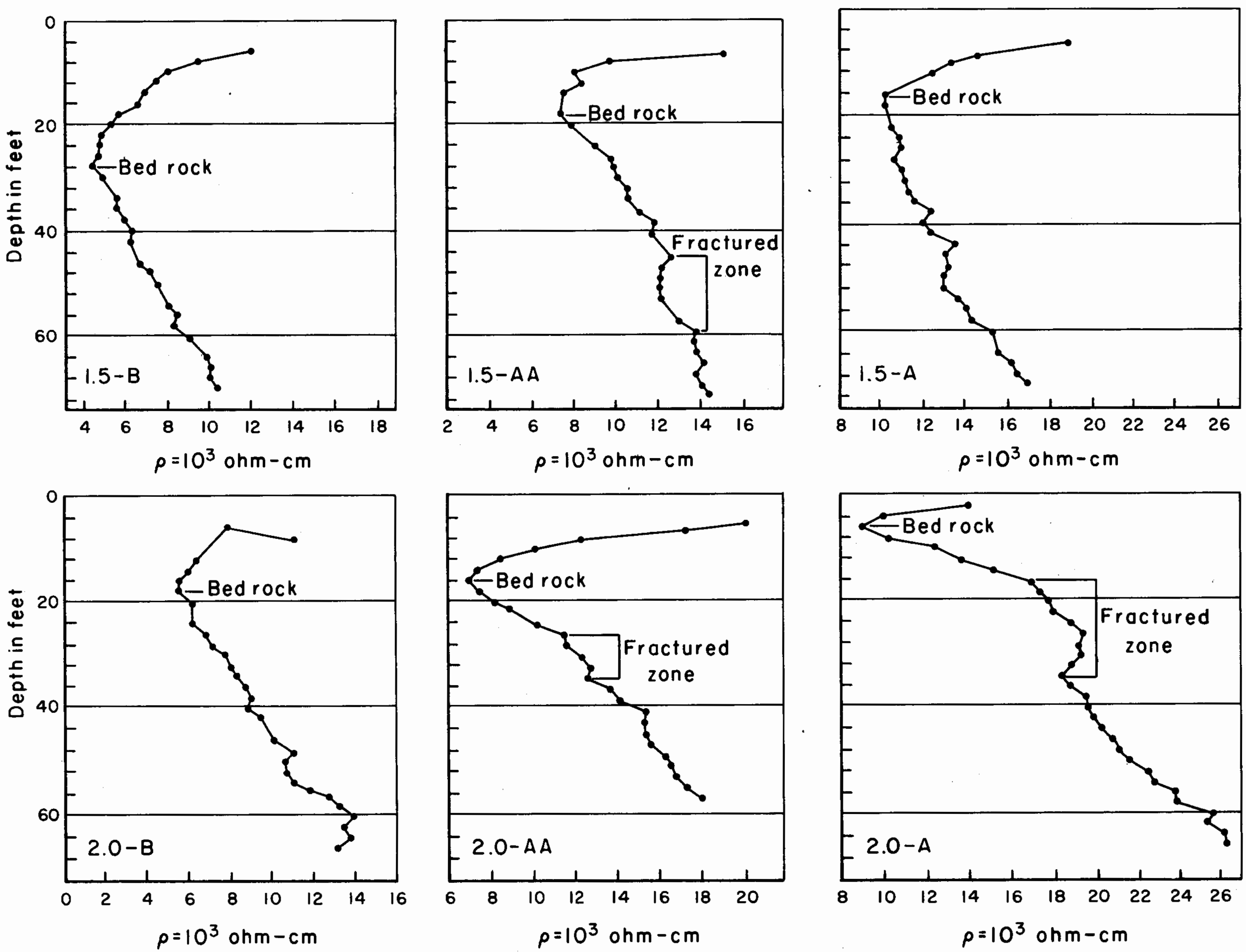


Fig. 31



ZENI SHAFT SINKING MACHINE
MODEL (X-153)

Fig. 32



R-341
3-26-59
291

Fig. 33 — Correlation chart showing method of delineating fractured zones in bedrock

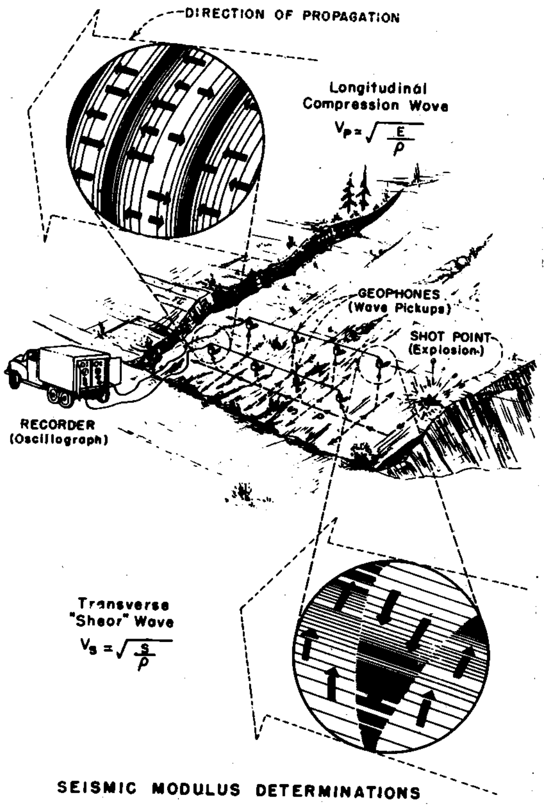

DIRECTION OF PROPAGATION
Longitudinal
Compression Wave
$V_P = \sqrt{\frac{E}{\rho}}$
GEOPHONES
(Wave Pickups)
SHOT POINT
(Explosion)
RECORDER
(Oscillograph)
Transverse
"Shear" Wave
$V_S = \sqrt{\frac{S}{\rho}}$
SEISMIC MODULUS DETERMINATIONS

Fig. 34

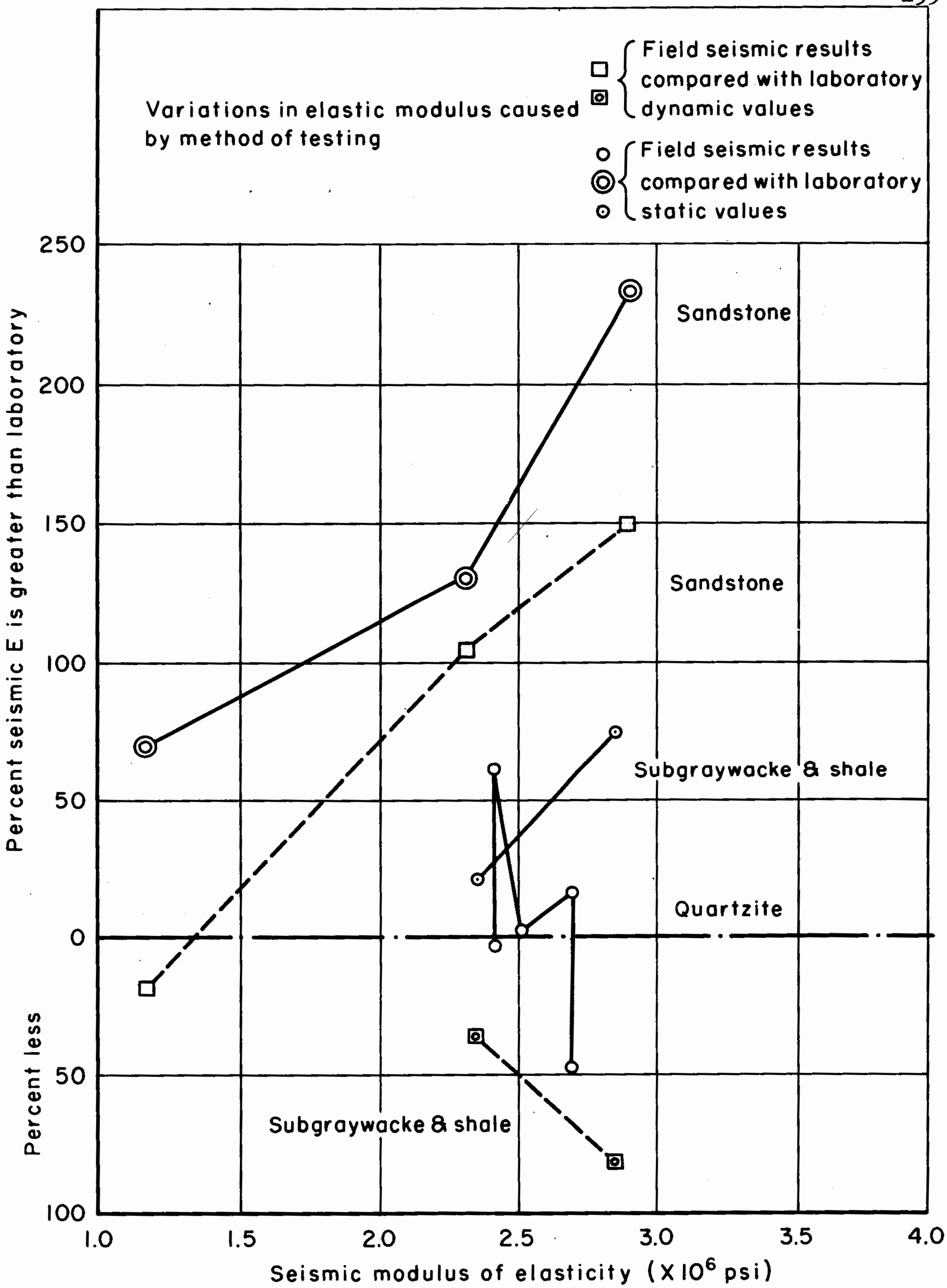

Variations in elastic modulus caused by method of testing
Field seismic results compared with laboratory dynamic values
Field seismic results compared with laboratory static values
Sandstone
Sandstone
Subgraywacke & shale
Quartzite
Subgraywacke & shale
Percent seismic E is greater than laboratory
Percent less
250
200
150
100
50
0
50
100
1.0
1.5
2.0
2.5
3.0
3.5
4.0
Seismic modulus of elasticity ($\times 10^6$ psi)

Fig. 35

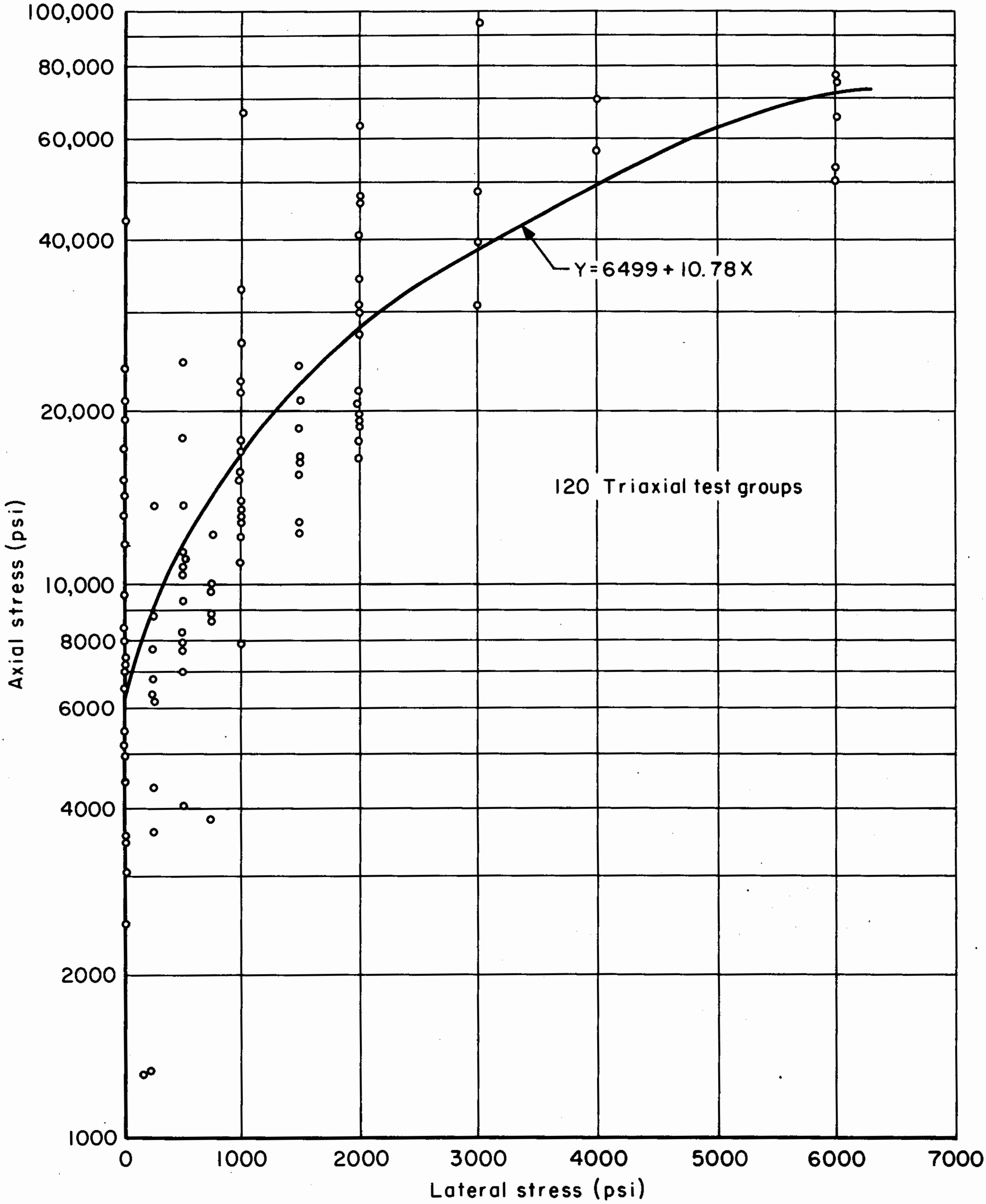

100,000
80,000
60,000
40,000
20,000
10,000
8000
6000
4000
2000
1000
Axial stress (psi)
0
1000
2000
3000
4000
5000
6000
7000
Lateral stress (psi)
Y=6499+10.78X
120 Triaxial test groups

Fig. 36

POTENTIAL APPLICATION OF BOREHOLE PHOTOGRAPHY TO SITE INVESTIGATIONS

Robert H. Nesbitt
Chief Geologist
Office, Chief of Engineers
U. S. Army

Yesterday we had a lucid presentation of the techniques to be followed in determining the suitability or unsuitability of underground sites. Today we have heard of some of the realities that the site selector faces. I was impressed with the pictures presented by Walter Douglas, particularly with that one site, in which many problems we will face in all sites were well reviewed. We might say (and I think Mr. Douglas will agree with me) that in that specific site we had better than average conditions. Perhaps a better example of the ideal site was the massive sandstone that Bill Judd showed for one of his tunnels, which comes as close to being a homogeneous material as we could expect in this type of work.

Now the geologist is faced with the job of using all the available literature and maps applicable to a site, executing an economical drilling program, and finding you a site which comes somewhere near the requirements which were set forth yesterday. He has to be sure that the drilling exposes all rock structure which is either favorable or unfavorable to the project. The groundwater conditions which could be a problem in the design of the structures must be disclosed early in the exploration, and in greater detail as it progresses. He has also to be certain, in the course of his investigations, that he has an adequate water supply for the underground installation. There are many other problems of that nature which these chaps, some of whom are here today, will have to answer as they start looking for sites. I am sure it is superflous to say that the ideal preliminary site study combines thorough aerial photoanalysis with a ground reconnaissance.

My comments will be confined to a tool which will aid the geologist, once a site has been selected, in disclosing conditions which affect the orientation of chambers and the location of tunnel portals. This instrument is the "NX" borehole camera. The Corps of Engineers has been using the camera since 1952, when we investigated a fault in the foundation of Folsom Dam, a condition which to a large degree eluded identification by the usual methods of foundation investigation.

The device surpasses other cameras designed for photographing interior surfaces of pipes, wells, and conduits in its capacity for obtaining continuous undistorted cylindrical color pictures of water-filled or dry borings.

The usefulness of the camera is at once apparent. To the geological, mining, and civil engineer alike it signifies the end of considerable guesswork with respect to underground conditions which could not be identified by conventional core-drilling methods.

Research and development for the first camera and projection equipment were conducted by the Engineering Research Associates, Division of Remington Rand, Inc., under the technical supervision of, and in accordance with plans and specifications furnished by, the Geology Branch, Office of the Chief of Army Engineers in Washington. Subsequent modifications of the camera lowering device, transporting equipment and special field tools were made in the Corps of Engineers' Ohio River Division Laboratories in Cincinnati. The background of this development is interesting.

Among the more difficult foundation problems involved in the construction of concrete dams and underground structures are those which result from bedrock imperfections that escape disclosure by conventional exploration methods.

These imperfections frequently are of sufficient magnitude to introduce costly changes, especially when their presence is discovered only after foundation excavation is well in progress. Bedrock flaws most frequently camouflaged may be ruptures resulting from the enormous stresses to which rocks have been subjected, or they may be planes or zones of weakness related to chemical alteration and underground erosion. Obviously, detailed information on their location, dimensions and structure is important, as these factors can affect the stability of a structure and the security of life and property in its environment. Their discovery and appraisal entail extensive subsurface exploration.

At present the most economical exploratory tool available to geologists and engineers is the small-diameter diamond core drill by means of which cores of the foundation bedrock are recovered for examination and testing. All too frequently, however, these samples fail to disclose minor foundation flaws that may be of major importance before a structure is completed. As a result, the small diameter drillings are supplemented generally by more reliable, but much more costly, shafts, tunnels, or large-diameter calyx drill holes which permit the investigator to examine the foundation rocks in place. It was to reduce the number and cost of these expensive and time-consuming investigations, as well as to obtain more complete information on rock structure from the smaller borings, that the "NX" borehole camera was developed. That it has been eminently satisfactory in accomplishing that purpose is demonstrated by its success on a large number of flood control, navigation and power projects of the Corps of Engineers. On a number of dams it has been used before and after construction, to determine the effectiveness of pressure grouting of foundation openings. On others borehole photography has been used to determine foundation grades, and length and orientation of foundation anchors.

In addition to locating and photographing a deep foundation fault zone at Folsom Dam, the pictures recorded delicate changes in rock coloring, fractures as small as 1/100 of an inch, and the surface of the groundwater table. A compass, visible in each picture, permits orientation of all these features on a unique cylindrical projection screen. Since the materials in fault zones are generally too soft to be recovered completely by the small-diameter core drill, their investigation, and the orientation of the fault, in the past could be accomplished only by tunnels, shafts, and costly man-size calyx borings. At Folsom the camera came to the rescue in time to eliminate a number of calyx borings by producing faithful color-picture records of some twenty small-diameter holes. The camera has also been used successfully in locating cracks in large concrete structures.

Externally, the camera unit is a simple stainless-steel tube (Fig. 1) 33 inches long and 2-3/4 inches in diameter, with a circular quartz window (e) not unlike a miniature lighthouse window, located five inches above the lower end. Internally, the principal elements include an oil-damped compass (o) which supports a hollow and truncated conical mirror (n). The latter is situated inside and directly opposite the quartz window, and thus comprises the "eye" of the camera. The hollow and truncated construction of the conical mirror makes the underlying compass visible to the 15 mm camera lens (k) located a short distance above the mirror. A high-voltage circular flash-tube (m), midway between the cone mirror and camera lens and actuated by a current-pulsing device on the camera lowering rig, simultaneously illuminates the boring and exposes the film to the bright mirror image. Directly above the lens is a conventional 16 mm moving-picture camera (h, i, j), with motor and spool drive synchronized by the same pulsing circuit which actuates the flash tube. A power condenser and relay unit (g) contacts the top of the camera unit. The camera is designed to make 16 flash exposures per foot of boring. Thus with a 1-inch section of drill hole exposed at each flash and with 16 exposures per foot of boring, sufficient overlap from picture to picture is obtained.

Suspending the camera from the ground control mechanism is a three-conductor cable, armored externally by reverse-lay, preformed, steel wire which prevents cable twisting. The tensile strength of this cable is 2700 pounds.

On the ground surface the camera is controlled by a lowering device (Fig. 2) which consists of a pay-out reel (c) with a capacity for 500 feet of camera cable, level-wind control (b) for the cable guide reel (h) geared to a 16-notch pulsing wheel and depth counter (i and j), hand-crank and gear drive (g) to the cable pay-out reel for lowering and raising the camera. Attached to the lowering mechanism is a small power supply cabinet (a) which transforms 117 volts a.c. to three d.c. circuits of 450, 50 and 50 volts, respectively. The highest voltage is transformed to 15,000 volts in the flash tube circuit of the camera; one 50-volt circuit is connected to the camera motor drive and synchronizing relay; the other 50-volt circuit provides the synchronizing pulse. When a film is completely exposed, the individual pictures are projected and studied as a series of separate still photoflash images, and not as a moving picture film.

Pictures are taken during the ascent or retrieving of the camera, inasmuch as cable tension is more constant and the motion of the camera steadier. A "dummy camera" of the same weight and dimensions is lowered and retrieved before risking the real camera in the boring. Since the camera pulsing-wheel is geared into the crank and gear-drive of the cable reel and guide reel, the time interval between picture exposures is controlled by the speed at which the hand crank is turned. Best results are obtained when the exposure interval is 2/3 of a second. With a capacity for 25 feet of 16 mm film, 75 feet of boring may be photographed continously before reloading is necessary. Later models will have a film capacity for photographing 100 feet of boring.

The camera is designed to withstand a hydrostatic head of 500 feet. Pictures obtained in dry and water-filled borings are equally good.

All of the camera equipment is portable into any site where diamond core drilling equipment can enter. With the aid of temporary cribbing and leveling screws (d) attached to the frame, the ground control mechanism can be leveled and made ready for operation a few minutes after uncrating of the equipment at the site. A portable 117-volt a.c. gasoline generator supplies the necessary electric power, when other sources are unavailable.

Figure 3 shows the device on which you examine the pictures. It is a modified Eastman Recordak Model "PM" microfilm viewer. On screen PS you can see a plane image about 3 in. in diameter. There is also the cylindrical viewing screen CS. Located on top of the cylindrical screen is an adjustable azimuth scale which rotates as a table and has a central circular window with two parallel lines between which the compass position of each picture may be aligned. By rotating the azimuth table until the compass arrow is centered between the window lines, the orientation of each feature on the cylindrical projection is obtained.

When a spool of film is mounted at FS, you just turn on switch FD and the pictures start moving. They can be seen on screen PS when mirror AM is in recumbent Position 1, on cylindrical screen CS when AM is in the 45° Position 2.

Picture detail is such that you can see fractures of a hundredth of an inch either in plane image as the lens sees it, or in cylindrical form. Figure 4 is an artist's exact reproduction of the plane and cylindrical image pictures of a minutely fractured zone in granitic rock. The 15.25 cm (5 in.) diameter of the cylindrical screen permits the viewer to examine each cylindrical picture of the borehole at approximately double scale. Inasmuch as considerable light is lost in projecting the film image through the conical prism and thence to the cylindrical diffusion screen, the latter is used solely for reading the dip and strike of geologic structure and for obtaining the undistorted dimensions of various textural and structural details. For the majority of his borephoto logging the geologist uses the plane screen where bright natural-color pictures, showing minute structural and textural details, can be analyzed.

I like this picture (Fig. 4) because it is an example of many instances in which similar zones of rock were not recovered in drilling, and consequently labelled as "void" or "opening." In this particular case fracturing which prevented core recovery extended for two feet. Until the camera was used the geologist could not be sure whether he was dealing with a fault, rock opening or fracture zone. The camera revealed a condition entirely amenable to consolidation by cement, and consequently a satisfactory foundation without deeper excavation.

Figure 5 is another example of the detail which the photographs will show. This is a limestone from Missouri with an interfingering bedding plane. The inside of the "doughnut" is the top of this conical mirror view. The outside of the "doughnut" is the base of the conical image. The conical mirror of the camera is a periscope in the hole. They eye looking down on this mirror would give you a doughnut-shaped image of this nature. The figure shows the wavy bedding structure, which was sheared off in the core. Here, it is preserved perfectly.

Figure 6 is the complementary cylindrical image of the wavy bedding keys seen in Fig. 5. Boring operations, of course, sheared the keys, and the detail interlocking of one bed with the other, which is important in structural considerations, was totally obscured. Here it was obtained with a relatively simple device.

Figure 7 shows how effective our foundation grouting was at Table Rock Dam in Missouri. I am sorry I do not have the pregrouting views. If we had a color slide, a pink tinge which we put into this grout could be detected in the darker areas. These are diagonal fractures dipping at 35 degrees from the vertical, about 35 ft below the base of the dam. The closeup study of this and similar pictures showed the original openings to be thoroughly grouted. Prior to the camera, core drilling to check the effectiveness of grouting was not always reliable. Figure 8 is the complementary cylindrical view.

Yesterday we saw a possible application of borephotography in connection with determining the effectiveness of the grout seal placed behind the saucer-shaped antenna structure for an underground communication center.

Perhaps the greatest contribution of the new camera is its capacity to determine the orientation, structure, dimension, and composition of rock fractures, and to identify these features early in the investigation. The core samples, taken from 3-in. holes drilled with the best supervision, often leave the geologist in doubt as to whether sample losses are the result of shear resulting from drilling, or natural openings and cleavage not related to drilling operations.

For the design of large structures knowledge of the in situ condition of the bedrock is extremely important. Until we developed this camera, the only device we had was the man-size calyx hole, illustrated at the conclusion of the last paper. These large borings cost anywhere from $50 to $200 a foot, whereas the "NX" hole can be drilled for $5 to $15, depending on the depth. The cost of photography in such holes ranges from $1 to $3 a lineal foot.

The geologists in the Corps of Engineers who have been trained to use this camera have also been trained to discern conditions in the photographs which indicate the need for supplementing borephotography with exploratory shafts or tunnels. The camera cannot solve all problems. It has saved the public many thousands of dollars as an exploratory tool during the past five years. Its use is routine on many of the Corps of Engineers projects at the present time. The patent is owned by the Government.

DISCUSSION

MR. LARS DE JOUNGE (Sandvik Steel, Inc.): What is the price?

MR. NESBITT: I am glad you raised the question of price. I could give you the manufacturers' names if you want to jot them down. One is the Engineering Research Associates, Division of Remington Rand, Inc., St. Paul, Minnesota. The other manufacturer, equally interested in getting invitations to produce cameras, is the Republic Engineering and Manufacturing Company, Snelling Avenue, St. Paul, Minnesota.

Now, I cannot give you the cost. The cost of the development of this successful camera was in the order of $20,000, a great amount of which was research. The optical elements of the camera are the most expensive. Perhaps there are $2,500 to $3,000 worth of optical and electronic materials in the camera and lowering device alone.

I am not quoting a price because this is not an assembly-line job yet. The first few that will be delivered will naturally come for a higher price. I do think that if the number of orders increases (and we currently have inquiries coming in from all parts of the world: Australia, South Africa, and other areas), such increase could favorably affect the price.

BRIGADIER GENERAL J. E. GILL (ADC, Colorado Springs, Colorado): You talked about going down vertically. Can you go horizontally?

MR. NESBITT: That is a good question. I mentioned the importance of horizontal and oblique holes. When using oblique holes, the camera's compass will operate at angles up to about 22 degrees from the vertical, and it will be modified to work at steeper angles. It is limited to 22 degrees from the vertical right now. That is the maximum at which the compass will function. That doesn't mean you could not use the camera in a horizontal hole for obtaining some information on the dimension and spacing of joint planes. In such cases the camera could be forced to the end of the bore with rods, and retracted in the usual manner. This is a minor design problem which we have asked both these manufacturers to iron out. They can have a compass in the next camera flexible enough to read any angles between vertical and horizontal. It is very important that you drill oblique holes for the camera if your rock structure and cleavage are such that vertical holes are inadequate.

BRIGADIER GENERAL GILL: You still need gravity.

MR. NESBITT: Yes, for vertical or oblique holes gravity, controlled by the cable wheel, lowers the camera. You are working against this weight as you bring it out, which makes for a smoother operation. We invariably lower the camera to the bottom before taking pictures, run a few feet of leading film off, and then proceed with the photography.

The camera control device has a depth counter, as does the projector, which counts in hundredths of feet. This little film box, right here, says, "Chief Joseph Dam, dummy 260.55' to 180.55'; camera, 260.00 to 180.55'." When I put this on the projector, I set the counter at 260.00 feet, and I read picture depths in ascending sequence as the views go past that projection screen automatically.

The projector, incidentally, is a modification of Eastman Kodak's Recordak PM Microfilm Viewer. You can use it with or without the cylindrical screen which, in earlier periods of picture analysis, is an aid in orienting structure and learning the relationship between the cylindrical and plane image projections. We have used the camera to trace grouting in foundations and minute structure in all kinds of rock.

FIGURES

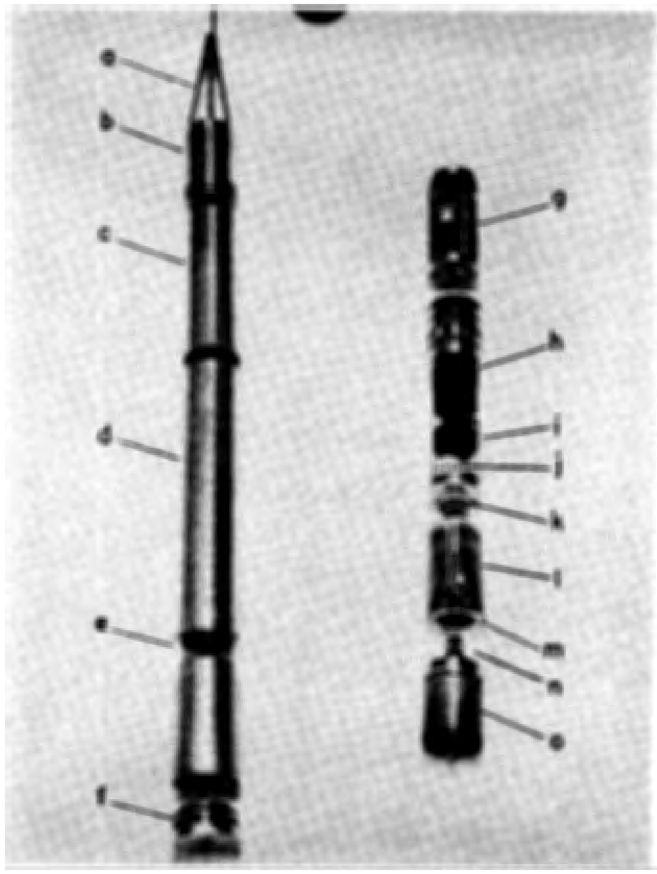


Fig. 1

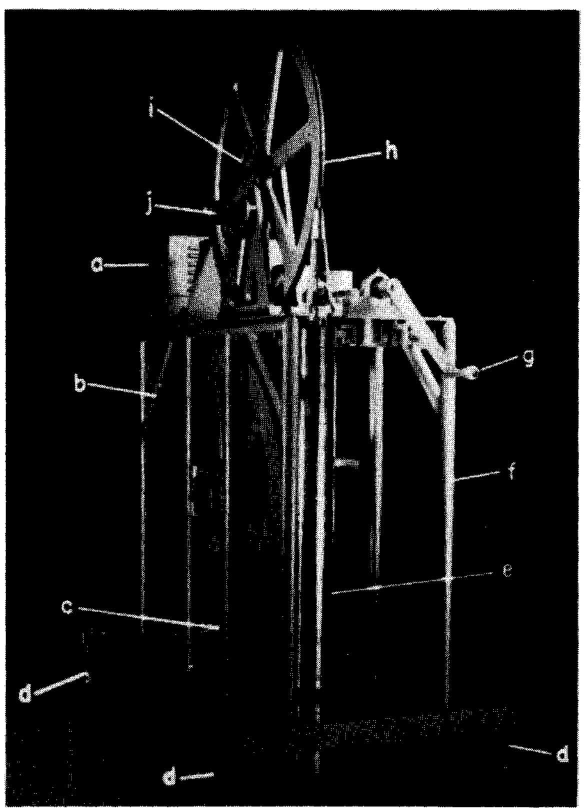


Fig. 2

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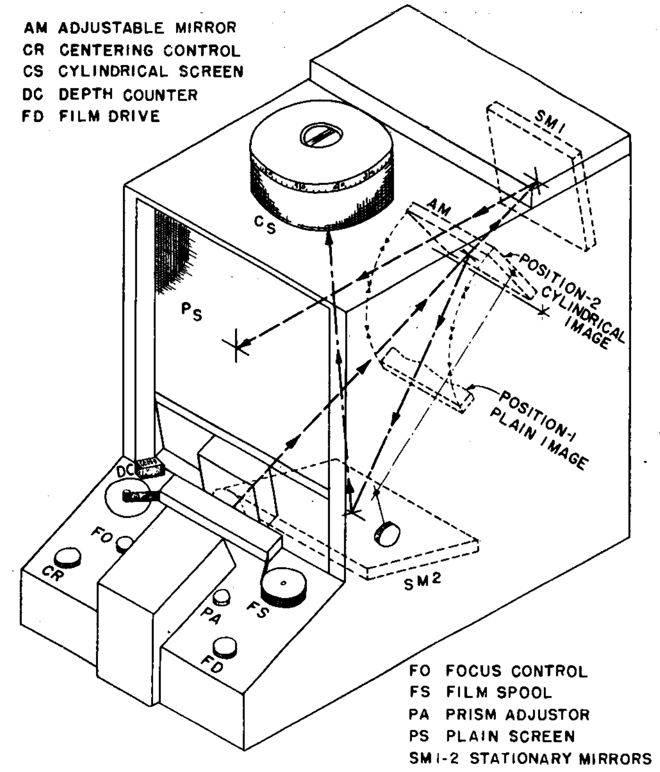
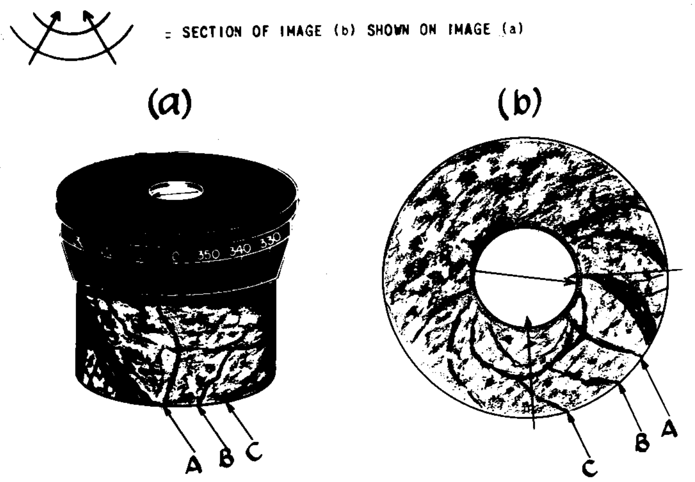
310



BOREHOLE CAMERA FILM PROJECTOR

Fig. 3

BOREPHOTOGRAPH (ARTIST'S CONCEPT)



Cylindrical (a) and plane-image (b) bore photograph of undisturbed fracture detail in gneissic granodiorite, 80 ft. below foundation of Chief Joseph Dam, Columbia River, Washington, U. S. A. Image (a) two-thirds natural scale. Note reversal of fracture positions and dip resulting from prismatic projection of film image (b) to cylindrical image (a).

Fig. 4

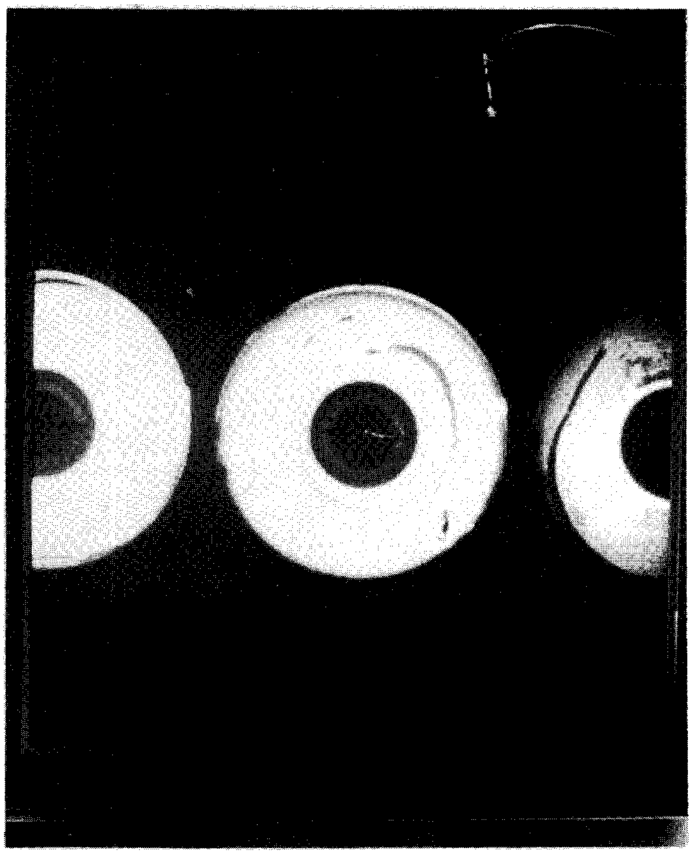


Fig. 5

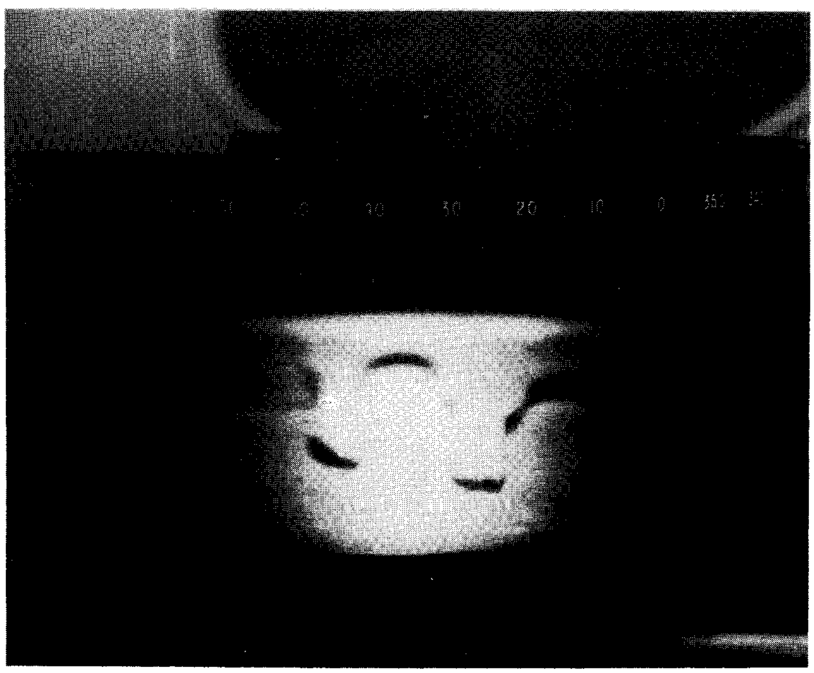


Fig. 6

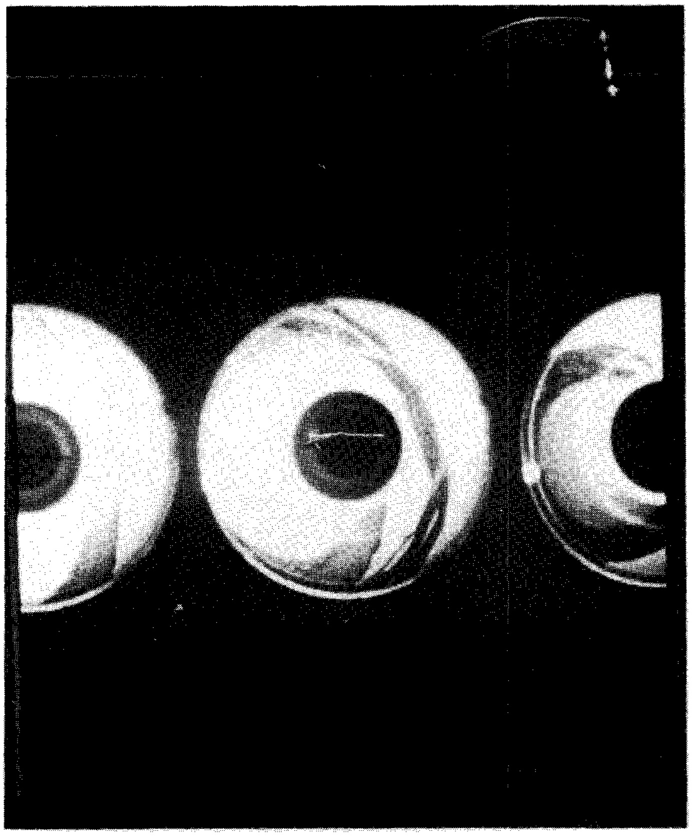


Fig. 7

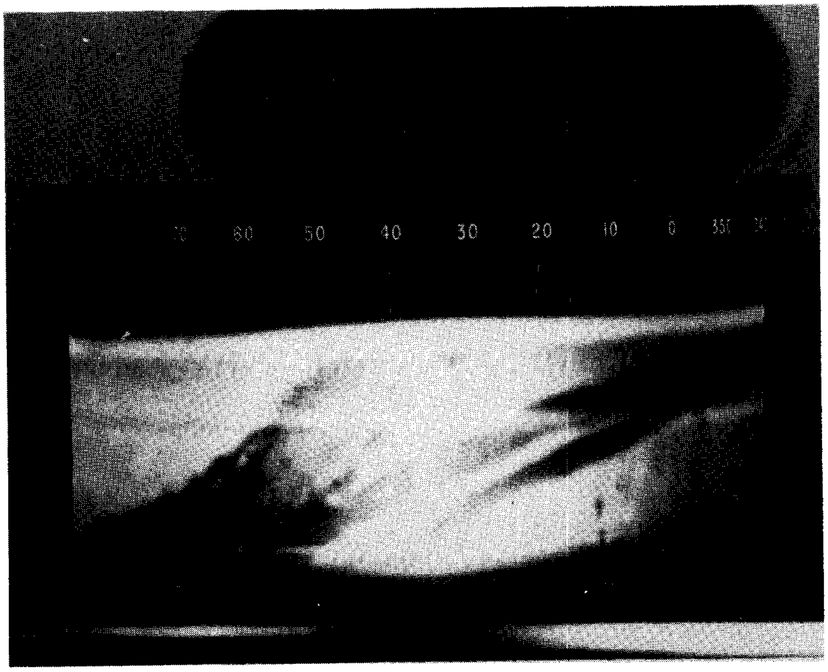


Fig. 8

TUNNEL DAMAGE FROM NUCLEAR EXPLOSIONS

Sherwood Smith
Holmes and Narver

As many of you may know, Holmes & Narver has the responsibility for engineering at the Nevada Test Site of the U. S. Atomic Energy Commission. In this capacity, there has been a requirement for predicting the distances at which there would be various degrees of damage to tunnels from underground nuclear explosions and for design of linings which would provide for safe re-entry following tests. Last fall, preparations were being made at the Nevada Test Site for the series of tests known as Operation HARDTACK-Phase II. At that time, there had been only one nuclear explosion in rock. This was the Rainier event at the same location.

Since Rainier constituted the principal basis for establishing damage-distance criteria for HARDTACK-Phase II, I will describe briefly the results of this test. It was planned and conducted by the Lawrence Radiation Laboratory (then called the University of California Radiation Laboratory). The device had a yield of 1700 tons or 1.7 kilotons of high explosive and was exploded on September 19, 1957, at the end of a tunnel driven into the side of the mountain. The self-closing configuration at the firing point is of particular interest and is shown in Fig. 1. You will see that the distance between the firing chamber and the nearest rib of the tunnel is 25 ft. The tunnel at this point was designed to be closed by the rock pushed outward by the explosion before the blast could travel the greater distance around the loop. This configuration achieved its purpose.

The geology of the mountain and the profile of the tunnel are shown on Fig. 2. The explosion occurred in TOS_7, which is described as "bedded tuff, mostly loosely cemented and 'sandy.'" Of general interest are the cap of welded tuff or rhyolite about 250 ft thick, the various layers of bedded tuff and the one layer of welded tuff, TOS_6, and the limestone basement rock, Dd. The vertical depth of burial was 899 ft and the nearest point on the sloping face was 790 ft. The tuff weighs about 125 lbs/cu ft and has a compressive strength of 5000-10,000 lbs/in^2.

The explosion created a cavity 55 ft in radius and the rock was crushed out to a radius of 130 ft, as determined by subsequent borings. The theoretical shock pressure vs distance is shown in Fig. 3. The indicated limit of crushing would be slightly less than 1000 (bars) or about 14,500 psi. This is greater than the strength indicated in laboratory tests of a specimen without lateral support. The fact that the material would have a degree of lateral support in place and the fact that rock in common with other materials probably has higher strength under high rates of strain may account for the difference.

The tunnel was about 7 ft high and 8 ft wide and of rectangular shape with rounded corners. At a radial distance of 200 ft it was completely collapsed, as shown in Fig. 4. Spalling continued to a distance of 400 ft and the condition at this point is shown in Fig. 5. Beyond 400 ft, the amount of spalled material amounted to only a few cubic yards and there was no damage beyond 500 ft. The corresponding scaled distances for a 1 kiloton yield are obtained by dividing the above distances by the cube root of 1.7. On this basis, no lining to resist explosive damage would be required at a scaled distance greater than 420 ft. At scaled distances less than 420 ft, linings would be required with progressively increased strength. For explosions of other yields, the distances would be multiplied by the cube root of the yield in question to predict corresponding damage.

Fortunately, results of an extensive series of high-explosive tests in sandstone, granite, and limestone were available. These were conducted by the Corps of Engineers in Utah and Colorado from 1948 to 1953 with the assistance of the Colorado School of Mines, the Bureau of Mines, and the Engineer Research Associates of St. Paul, Minnesota. There were tests at several scales with a maximum weight of charge of 320,000 lbs of TNT at full scale. The corresponding horseshoe tunnel was 30 ft high. No linings were used in any tunnels. Information on quantities of rock broken from the tunnel surfaces and kinetic energies imparted to the rock were available from these test reports. It was appreciated that there were differences between the effect of nuclear explosions and those from TNT, but it was felt that by a correlation of high-explosive data from tests in sandstone with nuclear data from Rainier, useful criteria could be provided. On this basis, design loads for tunnel linings at several scaled distances were developed in terms of quantities of rock detached from the back and ribs by the explosion and the kinetic energies imparted to the rock. It was realized that there would be heaving of the floor but it was felt that it would be easier to muck out this material and re-lay the track than to attempt to resist this action.

The following principles were considered to be important in the design of tunnel linings for re-entry:

(1) The lining should be a yielding type which would be less likely to lose its supporting capacity by buckling or brittle failure.

(2) Blocking or cribbing should be used to support the rock and resist its detachment rather than have it thrown off and strike the lining.

(3) Greater support would be required where unusual geologic situations were encountered.

It appeared that no practical linings could be designed to survive at a scaled distance of less than 168 ft, corresponding to the 200-ft distance for Rainier, at which there was complete collapse.

The details of the design criteria and procedure developed from the high-explosive tests have a military classification and cannot be discussed here. The design employed for the larger drifts is shown on Fig. 6. As you see, steel continuous rib-type sets were used, held in position with spreaders. These were 6-in. wide flange 20-lb sections. Wood lagging, 3 x 12 in., was placed outside the sets or between the flanges. Blocking was specified all around on 2 - 6-in. centers. The spacing of the sets was varied to provide the required strength to support the broken rock and resist the impact forces at selected scaled distances from the explosion. Beyond a scaled distance of 420 ft, no blast damage was anticipated. However, in some cases safety during construction and safe use following completion required that a lining be provided. Therefore, the same sets were employed 6 ft on centers with lagging at the back and as required for the ribs.

Figure 7 shows the design for the smaller drifts. In this case sets were 4-in. wide flange 13-lb sections.

Having designed the tunnel linings according to the best available information with many questions remaining as to the best criteria, it was apparent that the knowledge in this field should be improved by participation in the tests. Accordingly, a test project was developed for observing effects on the tunnels and recording response of the sets. This was Project 26.13, "Evaluation of Nuclear Effects on Tunnel Support Structures," with Mr. A. A.

Lee, Mining Engineer, of our organization supervising the project. This participation was under the general direction of the University of California Radiation Laboratory with Dr. Roger Preston as Program Director. The project was approved by the Test Manager on September 3, 1958, and accelerated preparations were made for participation in HARDTACK-Phase II. In general, the following was planned:

(1) Determination of damage and changes in alignment by before-and-after observations and surveys.

(2) In unlined drifts, observation of rock movements and determination of fly rock velocities by high-speed motion picture photography.

(3) Measurement of transient deflection of selected sets by use of deflection gages and by high-speed motion picture photography.

The transient measurements yielded some qualitative information but the motion picture photography was not successfully accomplished for reasons beyond the control of the engineer responsible for the project. The damage observations, however, will be extremely useful and some of the more interesting results will be described.

The Logan event on October 15, 1958, provided more interesting damage information than any other test in the series. The total energy release is estimated at 4.5 kilotons ± 1 kiloton. The plan and profile of the tunnel are shown on Fig. 8. The device was exploded at the end of U12e.02 with 932 ft of rock cover. Four sandbag plugs were used as shown, and the lack of the self-closing configuration used for Rainier appeared to result in greater damage in the direction of the main drift. This would not be unexpected as the four sandbag plugs could hardly be as effective as the Rainier configuration in assuring a symmetrical effect. The actual asymmetry will not become known until free-field measurements of effects have been reduced and analyzed.

Figure 9 shows the general plan of Tunnel U12e. The drift used for Logan, U12e.02, had no lining as there was no requirement for immediate re-entry. Following the explosion, this drift was completely collapsed up to the junction with the main drift and beyond into the main drift to Station 19+60. This damage is shown in Fig. 10. Referring again to Fig. 9, a re-entry drift has now been driven for recovery of scientific equipment to a point 240 ft from the firing point. The face of the excavation in the old U12e.02 drift is of great interest. The appearance is that of a newly excavated face. There are no cracks and the rock is compressed so as to form a solid mass, at least equal in density and strength to the original. Sandbags could be seen in the face in sections and the soil in the bags had been compressed so as to resemble soft sandstone. Unfortunately, photographs are not yet available.

A stack of lead bricks had been placed in front of the alcove 156 ft from the firing point to shield equipment. This stack of bricks had been moved as a unit 90 ft down the tunnel and was found standing on end. There is no indication of melting of the lead but the bricks are fused so that the stack forms one welded mass.

Now let us consider damage to the main drift from the junction of U12e.05 toward the portal. The first 140 ft of the tunnel to Station 18+20 was very heavily damaged. The lining was designed for safety during construction only. Observation led to the belief that this section would have collapsed in a manner comparable to that for Rainier at a distance of 200 ft except for the support of the lining. The radial distance from the firing point to Station 18+20 was 820 ft. This would be a scaled distance of 500 ft compared with 168 for Rainier.

In this section, the rock with broken lagging and damaged sets was removed and a new lining was installed. The conditions following this work are shown in Fig. 11 at Station 19+20.

Severe damage extended from the outer limit of collapse about 150 ft towards the portal to Station 16+70. Sets were badly distorted and buckled portions extended into the opening. The obstructing portions of the sets were cut out by torch. Repairs such as the replacement of broken lagging, shoring, and blocking were accomplished as re-entry progressed. The rock in the floor had heaved, distorting the tracks. All broken rock was mucked out and a single track was laid in lieu of the two which had been there previously. For safety, a new lining was placed inside the old from Station 18+00 forward. In the remaining portion of the main drift, damage became progressively less toward the portal. This type of damage ended about 1200 ft from the firing point at Station 14+00.

Of particular interest, however, were two areas of damage which were closely related to the local geology. At Station 11+70, approximately 1900 ft from the firing point, the upper stratum of tuff moved along a bedding plane about 2 ft toward the portal with respect to that directly underneath. This is shown in Fig. 12. The movement resulted in telescoping of the track and lining, as shown in Fig. 13. A similar situation existed at the bedding plane at Station 6+50 at a distance of 2400 ft from the firing point. In this case, the slippage was about 3 ft.

The location of drift U12e.05 is shown on Fig. 9. This drift was approximately 9 ft high and 8 ft wide and was lined with steel sets and timber lagging as shown previously. This lining extended out to Station 6+40 and was heavily damaged as shown in Fig. 14. Lagging had been broken in many places and sets were distorted. The floor heaved from 1 to 4 ft and some rock dropped to the floor from the back and ribs. After mucking out, replacement of broken lagging and re-laying track, safe access was provided. Beyond Station 6+40 there was no lining and the drift was heavily damaged and nearly completely filled with debris, as shown in Fig. 15.

Pending further analysis, the unexpectedly heavy damage to this drift can be accounted for by the asymmetry of the explosion and the fact that the compressive pulse struck the drift from an angle approaching the perpendicular, which should be much more effective in producing damage than in the case where travel is along the axis of the drift.

The Blanca event occurred on October 30, 1958. The 23 kilotons $\pm$ 3 kilotons device was detonated in the U12e.05. Referring to Fig. 9, it had been planned to explode it at Station 9+14 but because of the partial collapse of the unlined section of this drift, it was exploded near the end of the lined section of the drift at Station 5+97. There was major collapse to a distance of about 850 ft from the firing point. At Station 19+30 in the main drift, the lining was squeezed into a space about 3 ft in diameter as shown on Fig. 16. For re-entry it was necessary to blast, excavate, and provide a new lining.

There was a further thrust of about 3 ft along a bedding plane at a distance of 1500 ft and additional movement at a distance of 2200 ft. These were the same locations at which slippages occurred in Logan.

Summarizing experience in HARDTACK-Phase II, I believe that the general concept of design of yielding-type linings for re-entry after nuclear tests was proven to be valid. In the Logan event, linings were probably exposed to loads greater by orders of magnitude than those for which they were designed. There was collapse in the main drift to Station 18 + 20. Otherwise, linings survived in severe damage areas without collapse. Repairs could be made with relative ease as the type of lining lent itself to such action.

The heavy damage from the Blanca event is interesting but is less significant in relation to design because of the changed location of the firing point.

Since the completion of the tests last October, we have been giving considerable thought to the improvement of design criteria. Fig. 17 illustrates the pattern of tangential stress around the opening with the pulse traveling down the axis of the tunnel. Stresses have been normalized so that the maximum combined static and dynamic tangential stress is equal to unity.

The static tangential stress was computed for the amount of rock cover with a horizontal component of one-third the vertical. The values of stress around the opening in terms of the free-field stress were obtained from the Corps of Engineers Manual on Design of Underground Installations in Rock. In the case of the dynamic stress pulse, the vertical and horizontal forces were taken as one-third the pressure obtained from Fig. 3. Using this as the free-field pressure in the plane perpendicular to the axis of the tunnel, the stress was computed for selected locations around the opening, using the above reference.

Since the critical area is at the bottom of the horseshoe where the section is rectangular, Rainier data would be applicable. At 400 ft where continuous damage ended, the pressure was 210 (bars) from Fig. 3. This would result in a tangential stress at the lower corner of 3920 psi. Adding this to the static tangential stress at the same location, 1715 psi, the total stress for incipient failure would be 5635 psi. This is consistent with the compressive strength of the rock from laboratory tests of 5000 to 10,000 psi.

A similar investigation was then made for the situation where the compressive stress pulse comes in from the side. The same static stress pattern would be applicable. The dynamic stress pattern was developed for a horizontal stress equal to the free-field stress and vertical component at back and floor equal to one-third of the free-field value. The maximum total tangential stress occurs at a point about one-tenth of the distance from the corner toward the center of the floor. Subtracting the static stress at this point from 5635 psi, the remainder is 4750. Dividing this by the appropriate factor for increased stress at this point, the corresponding free-field stress would be 1480 psi or about 100 bars. Extrapolating, the distance for this pressure would be 800 ft. In other words, for this material the distance for incipient failure is twice as great for the wave striking the tunnel at right angles as for the case where it travels along the axis. The above study is of general interest only but may be a beginning to the solution of the problem of relating free-field data and tunnel damage criteria. Some of the uncertainties are as follows:

(1) The actual value of the compressive stress pulse and its shape are not well known.

(2) With high strain rate, the rock in common with other materials probably has greater strength.

(3) The strength of the volcanic tuff varies through wide limits.

(4) The assumption of homogeneity used in developing stress patterns is not met in practice because of bedding planes, joints, and other varia-

tions which have a significant effect on stress and damage.

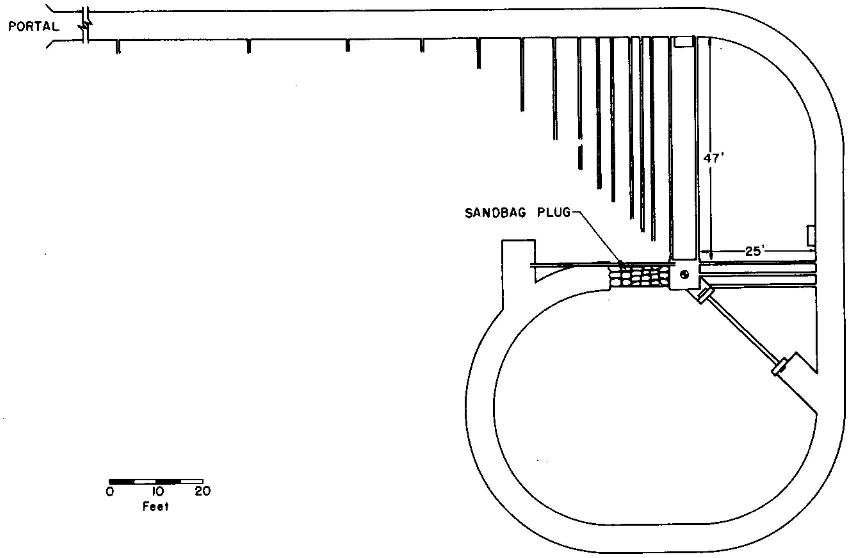
(5) Blasting for the tunneling operation does some damage beyond the limits of the excavation.

The stress pattern illustrated in Fig. 17 points up the possible benefit of a circular drift with a circular lining. Stress around a circular opening would be more uniform and the lining would have greater strength. From a construction viewpoint, however, this shape would be less convenient than the horseshoe shape.

Further information is needed on behavior of rock in unlined sections of tunnels when exposed to damage from nuclear explosions. This would provide better insight as to the requirements for design. Also, the relative performance of yielding and rigid types of lining of circular and horseshoe shape is required. If there are to be no more nuclear tests, high-explosive tests with model tunnels with proper charge scale relationship to obtain an appropriate wave length are a distinct possibility.

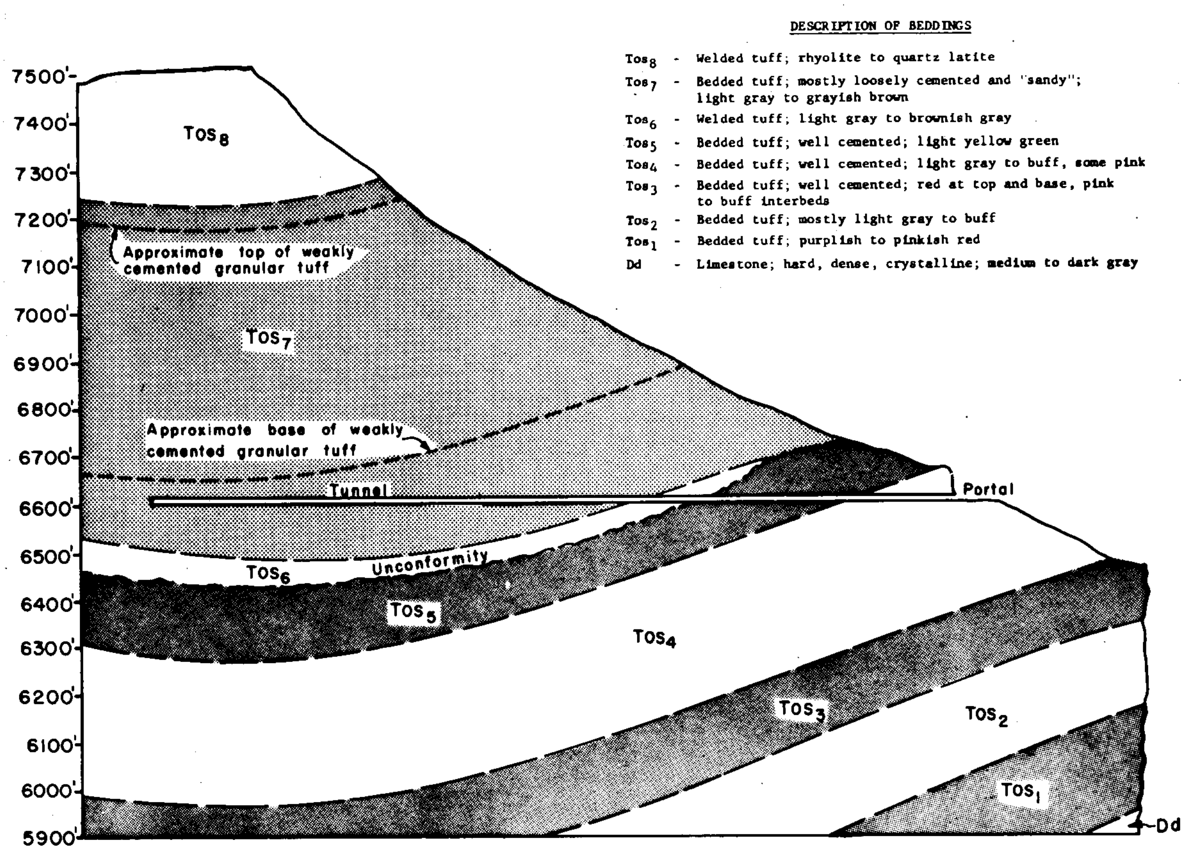
There is a great deal yet to be learned in regard to design of tunnels to resist nuclear explosions. Holmes & Narver was confronted with an immediate requirement for predicting damage and designing linings. Our experience in HARDTACK-Phase II contributes to the state of knowledge in this field and results were reasonably successful. Complete analysis of data in relation to free-field measurements, not yet available, should point the way to improved design.

FIGURES



Pre-shot tunnel configuration in region of detonation.

Fig. 1



Profile of tunnel site with geologic characteristics.

Fig. 2

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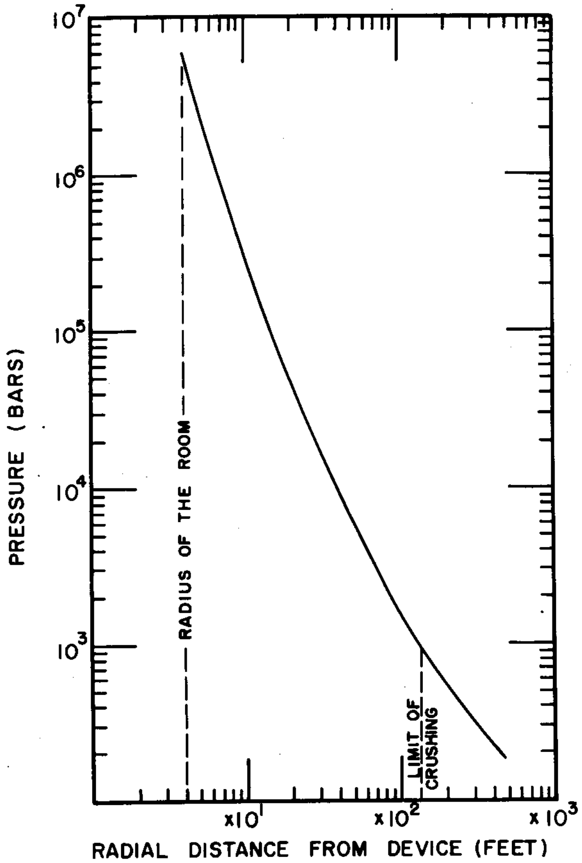
3-26-59
328



Estimate from theory of shock pressure vs radial distance.

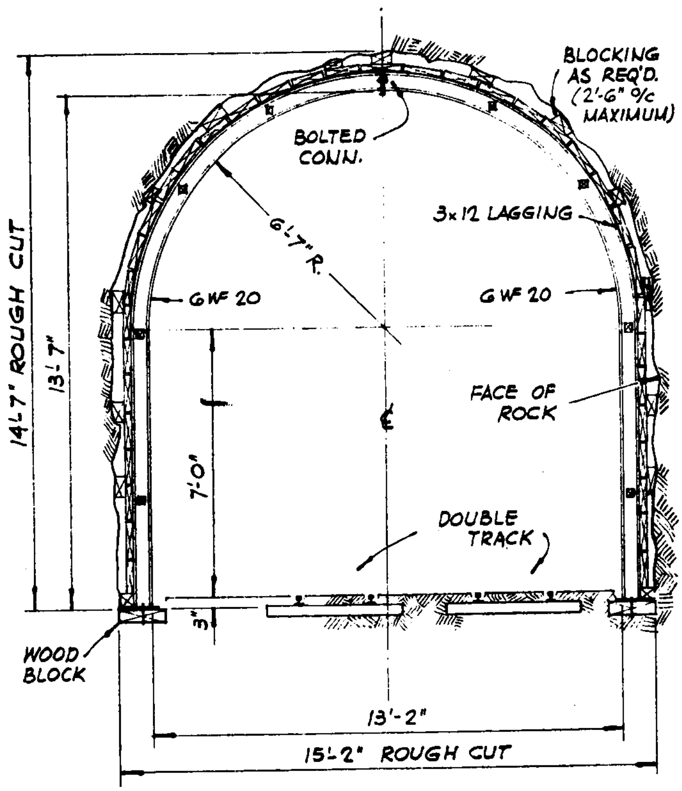
Fig. 3

COLLAPSE OF TUNNEL AT 200 FT
FROM FIRING POINT

Fig. 4

DAMAGE TO TUNNEL AT 400 FT
FROM FIRING POINT

Fig. 5



TUNNEL SUPPORT SET -U12e

Fig. 6

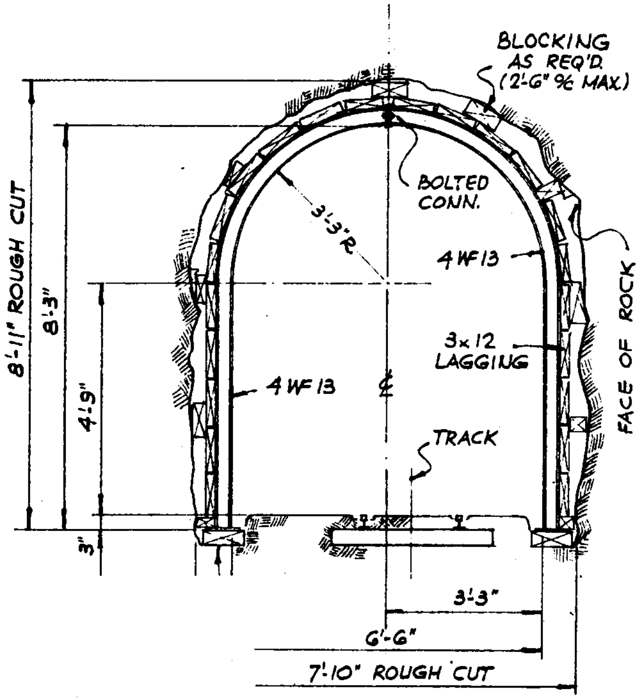

BLOCKING
AS REQ'D.
(2'-6" ℅ MAX.)
BOLTED
CONN.
3'-3"R
4 WF 13
FACE OF ROCK
3x12
LAGGING
4 WF 13
℄
TRACK
8'-11" ROUGH CUT
8'-3"
4'-9"
3"
3'-3"
6'-6"
7'-10" ROUGH CUT

Fig. 7

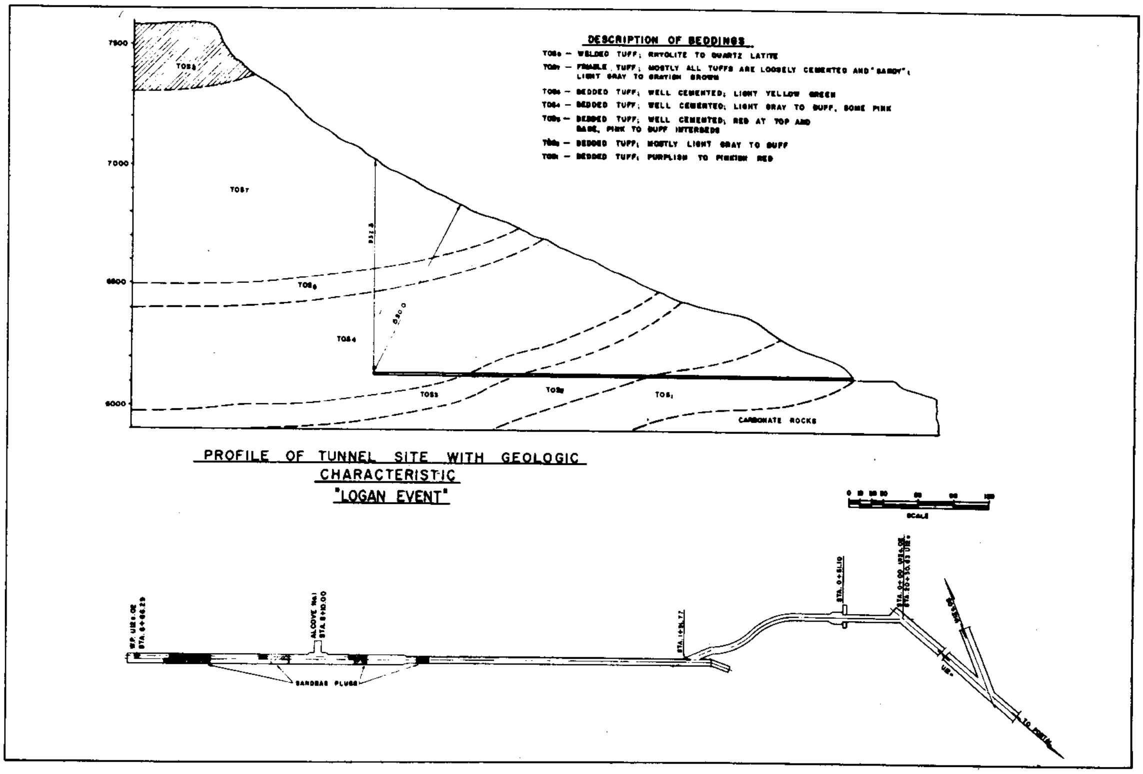

DESCRIPTION OF BEDDINGS
WELDED TUFF; RHYOLITE TO QUARTZ LATITE
FRIABLE TUFF; MOSTLY ALL TUFFS ARE LOOSELY CEMENTED AND "SANDY"; LIGHT GRAY TO GRAYISH BROWN
BEDDED TUFF; WELL CEMENTED; LIGHT YELLOW GREEN
BEDDED TUFF; WELL CEMENTED; LIGHT GRAY TO BUFF, SOME PINK
BEDDED TUFF; WELL CEMENTED; RED AT TOP AND BASE, PINK TO BUFF INTERBEDS
BEDDED TUFF; MOSTLY LIGHT GRAY TO BUFF
BEDDED TUFF; PURPLISH TO PINKISH RED
7500
7000
6500
6000
CARBONATE ROCKS
PROFILE OF TUNNEL SITE WITH GEOLOGIC CHARACTERISTIC
"LOGAN EVENT"
SCALE
SANDBAG PLUGS
TO PORTAL

Fig. 8

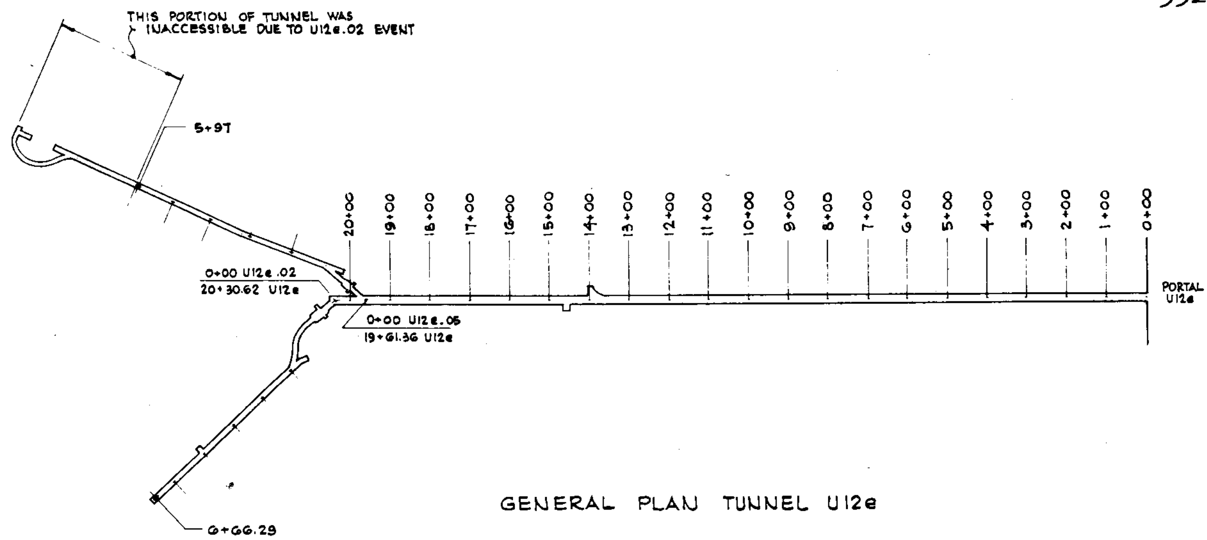

THIS PORTION OF TUNNEL WAS INACCESSIBLE DUE TO U12e.02 EVENT
5+97
20+00
19+00
18+00
17+00
16+00
15+00
14+00
13+00
12+00
11+00
10+00
9+00
8+00
7+00
6+00
5+00
4+00
3+00
2+00
1+00
0+00
0+00 U12e.02
20+30.62 U12e
0+00 U12e.05
19+61.36 U12e
PORTAL U12e
6+66.29
GENERAL PLAN TUNNEL U12e

Fig. 9

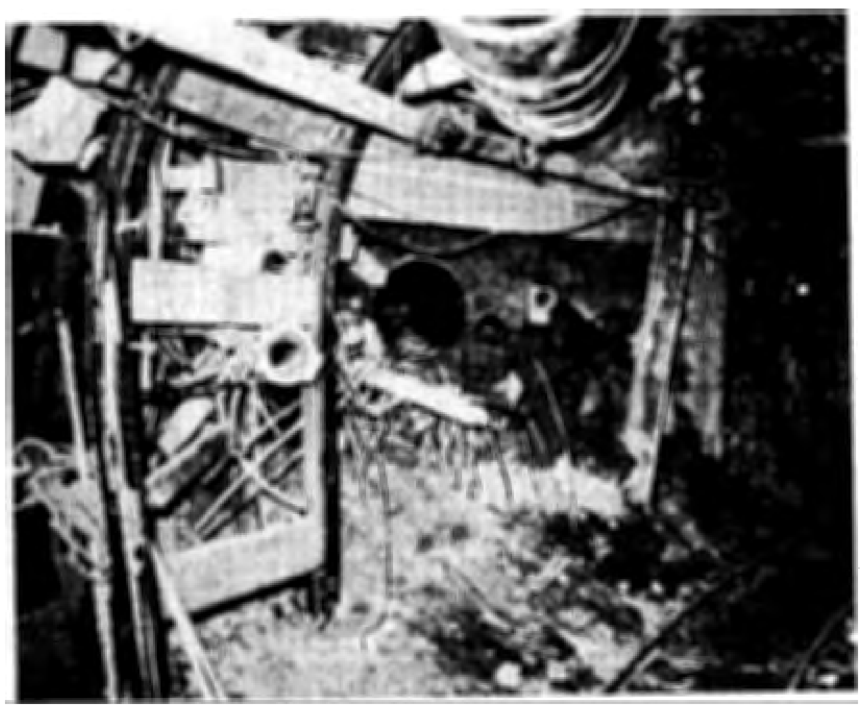


Fig. 10

Fig. 11

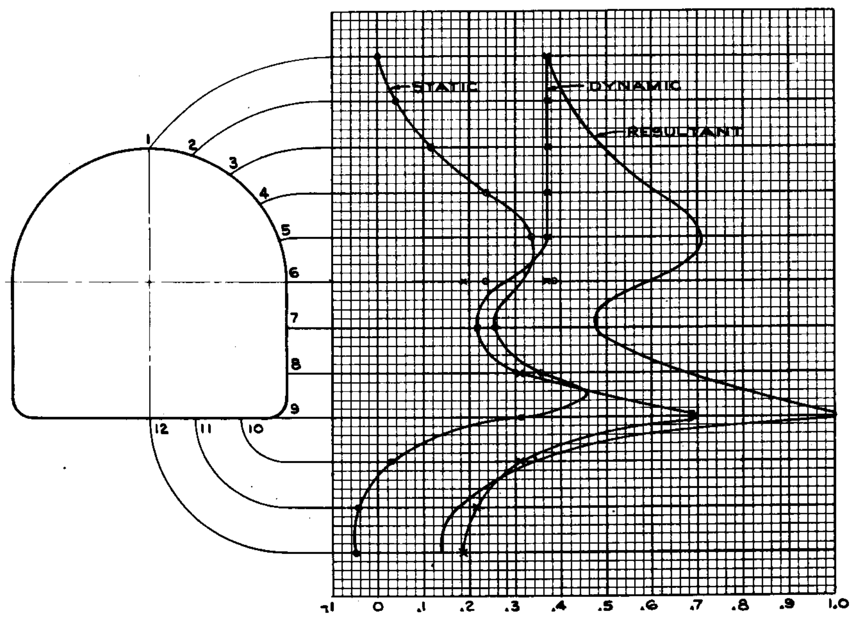
Fig. 12

Fig. 13

Fig. 14

Fig. 15

Fig. 16



TANGENTIAL STRESS PATTERN

Fig. 17

EFFECTS OF EARTHQUAKES ON TUNNELS

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Respectively, Professor of Engineering and
Associate Research Engineer,
University of California, Los Angeles

INTRODUCTION

An effort is made here to summarize and generalize the available information on earthquake damage to tunnels. Four reasonably well documented cases are presented, along with supporting and indirect evidence. The details of the failures and of the geology are less complete than is desirable, but the facts available appear to warrant several useful generalizations. The original sources cited may be consulted by those wishing to study the data in detail.

EXPERIENCE IN CALIFORNIA AND JAPAN

Central California, 1906. In the San Francisco earthquake of 1906 there were two damaged tunnels on the narrow-gage Southern Pacific Railroad between Los Gatos and Santa Cruz. The 6200-ft tunnel at Wright Station was crossed by the San Andreas fault and the 5700-ft tunnel directly to the south was also damaged, but to a somewhat lesser degree. Shaking at the surface over the tunnels was very intense, designated 10 on the Rossi-Forel scale. Damage to the tunnels themselves, Table 1, consisted of the caving-in of rock from the roof and sides, the breaking in flexure of upright timbers, and the upward heaving of rails and breaking of ties. Both tunnels were blocked at a number of points.

The tunnel at Wright Station suffered a 4.5 ft transverse horizontal offset where the fault cut it (Fig. 1). This same movement wrecked the Morrell house which stood above the tunnel and on the fault. Tunnel damage was greatest around the offset and at the several locations where parallel fissures were in evidence.

The rocks in the Wright tunnel looked like sandstones and jaspers of Franciscan age.

Other tunnels on the Santa Cruz-Los Gatos line were undamaged, except for two cases of broken timbers. New tunnels under construction on the Bayshore line, in southern San Francisco, were uninjured.

Tokyo Area, Japan, 1923. The great 1923 earthquake damaged about 25 tunnels, Table 2, in the vicinity of Tokyo, principally on the Izu and Boso peninsulas which are the mainland areas closest to the epicenter. The damage is attributed to shaking, as no case of faults intersecting the tunnels is known. Most of the tunnels were concrete or brick-lined, with depth of cover, character of rock, length, and other features varying over a rather wide range. Particularly heavy tunnel damage occurred in the Odawara-Atami-Hakone region, which suffered the highest intensity of shaking. Beyond the isoseismal corresponding to approximately 50 per cent of houses collapsed, tunnel damage apparently was insignificant.

Figures 2 through 6 illustrate the destruction in two selected cases. Damage varies from fractured portal masonry through cracked linings to cave-ins from roof and sides.

Tanna, Japan, 1930. The Tanna Tunnel, connecting Atami and Mishima, was under construction during the Izu earthquakes in 1930. The Tanna fault intersected one of the drain tunnels which extended ahead of the main tunnel heading, causing a transverse horizontal offset of 7.5 ft at a distance of about 2 ft beyond the main tunnel heading (Figs. 7 and 8). The only damage to the tunnel was a few cracks in the walls. But in the village of Karuizawa, situated on the Tanna basin 160 meters above the tunnel, 55 per cent of the dwellings were thrown down, and 40 per cent of the houses were destroyed at the nearby villages of Tanna and Hata. Surface displacements on the fault occurred over a distance of 15 kilometers.

The Tanna basin is a lake deposit of sandy clay and boulders, about 40 meters deep, overlying andesite and agglomerate through which the tunnel passes.

<u>Kern County, California, 1952.</u> The Kern County earthquake of 1952 severely damaged four tunnels, Table 3, on the Southern Pacific Railroad near Bealville, about 15 miles northwest of Tehachapi. This was the region of largest observed ground fractures associated with movement on the White Wolf fault (Figs. 9 and 10). In all, there were 15 tunnels between Bakersfield and Tehachapi, and those outside of but adjacent to the area of ground fractures suffered slightly, to the extent of opening of construction joints. The railroad in this area was built about 1876, with timber lining in the tunnels. Reinforced concrete lining 12 to 24 inches thick was installed later, without removing the timber. Rock around the four damaged tunnels was a fairly easily excavated decomposed diorite.

Tunnel No. 3, originally 700 ft long, was heavily damaged at its Tehachapi end, 200 ft of which was daylighted after the quake (Figs. 11 and 12). At one place the buckled rail extended under the concrete wall, indicating that the wall had raised sufficiently to permit this. While ground cracks were not found directly over No. 3, an active fault crossing the tunnel was found during daylighting.

Large surface cracks (Fig. 13) were found above No. 4, which was badly shattered (Fig. 14) and subsequently daylighted for its full length.

Tunnel No. 5 was very heavily damaged (Fig. 15) but was reconstructed without daylighting. Cracks and holes appeared in the ground above, and rock and soil from these cracks flowed into the tunnel.

Broken lining comprised the damage to No. 6 (Fig. 16) which was daylighted. No substantial surface cracking was noted over this tunnel.

These tunnels were in the region of heaviest shaking, Modified Mercalli Intensity XI, but clearly the extensive damage was primarily due to their location in the fault zone.

Fukui 1948 and Hokkaido 1952. At Kumasaka, north of Kanazu, the portal ar-arches of a brick-lined tunnel were partially fractured in the 1948 Fukui earthquake. Also in this earthquake, a large concrete culvert was badly cracked at midlength.

In the 1952 Hokkaido earthquake, minor cracking was induced in the walls of one concrete-lined and one brick-lined tunnel.

Fault movement at the tunnels was not involved in the above cases.

RELATED EXPERIENCE

Mines and Caves. During the Sonora earthquake of 1887, an engineer was in a mine at Tombstone, Arizona. He felt violent shaking and observed small rockfalls, but no collapsing, down to several hundred feet of depth. Most stopes were untimbered. Damage on the surface consisted of falling plaster and chimneys, and shifting of engines on their foundations.

In another case, in mines at Butte and Barker, Montana, the 1925 earthquake was hardly noticed by those underground but was felt at the surface.

The August 22, 1952, Kern County aftershock was reported not to have been felt by a party in Crystal Cave, Sequoia, but to have been sharply felt by persons outside the cave.

There appears to be a possibility that rock bursts at Widwatersrand, South Africa, may be triggered by releases of energy at points in the mine complex away from the bursts.

Experience of California Agencies. Correspondence from W. M. Jaekle, Chief Engineer, Southern Pacific Company, reveals that, except for the 1906 and the 1952 cases, no damage or disturbance to Southern Pacific tunnels has been caused by earthquakes.

The Los Angeles Department of Water and Power operates the Owens Valley Aqueduct, which includes 142 tunnels totaling 43 miles in length. The Aqueduct was completed in 1911, and no tunnel damage due to earthquakes has occurred. The Elizabeth Tunnel, five miles long, crosses 3000 ft of the San Andreas rift zone at a depth of up to 1000 ft. It is inspected annually; no earthquake damage has been found to date.

The Pacific Gas and Electric Company has experienced no significant earthquake damage to tunnels in its 40 years of experience with 73 tunnels, unlined and concrete lined, totaling 119 miles in length.

The above experiences are significant in view of the fact that California has experienced severe earthquakes in 1915 (Imperial Valley), 1925 (Santa Barbara), 1933 (Long Beach), 1940 (El Centro), 1952 (Kern County), and 1954 (Western Nevada), in addition to the great earthquake of 1906.

Effect of Depth below Surface. Several Japanese investigators have measured small earthquake motion at some depth and simultaneously at the ground surface. Omori (1902) was the first to make such measurements. Nasu determined the ratios of displacements of 14 earthquakes at the surface above Tanna Tunnel and in the tunnel at 160 meters depth to be 4, 2, 1.5, 1.2 for periods 0.3, 1, 2, 5 seconds, respectively. The ground was lake deposits at the surface and andesite and agglomerate at depth. Saita and Suzuki found that the maximum acceleration at the surface of a 68-ft layer of alluvium was three to five times that at its base contact with diluvium. Inouye found that short-period waves (ripples) observed at the surface were largely absent at a 9-meter depth.

Carder of the U.S. Coast and Geodetic Survey recorded approximately equal amplitudes of microseisms at the surface and at 5000-ft depth in Homestake Mine. Microseisms were of 4- or 5- second period. In a later study, earthquake P-waves of 1-second period were recorded at 300-ft depth with twice the amplitude recorded at 5000-ft depth.

Recently, Kanai has made signal progress in this field. He operated seismographs at depths of 0, 150, 300, 450, and 600 meters in a copper mine in Hitachi and recorded a very large number of small earthquakes. The ground is paleozoic rock, with some weathering near the surface. The ratio of surface maximum displacement to that at 300 meters depth was about 6 at the mine and about 10 at a school 6 kilometers away on alluvium. Earthquakes whose average period of incoming waves was close to the free period of the surface layer caused these maximum ratios, but many earthquakes occurred for which the ratios were as small as one-third of the above. He also found that the period of the short-period waves (ripples), found on surface seismograms but not underground, corresponds to the predominant period of the surface layer; the ripples dominated the surface record when the incoming waves contained components with period equal to that of the ripples. These findings support quantitatively the theoretical amplification formulas of Sezawa and Kanai.

Qualitatively, these researches demonstrate experimentally the following effects of depth:

1. At short periods, surface displacements are larger than underground displacements.

2. The ratio of surface to underground displacement depends on the type of ground. It is greater for alluvium than for weathered rock. It may reach a value of at least 10.

3. For wave periods over one second, the ratio becomes comparatively small, approaching unity as the period increases.

4. There is a particular average period of incoming waves for which a given type of ground will provide a maximum ratio of surface to underground displacement. If the average period of incoming waves is not approximately equal to this particular period, the ratio will be materially smaller.

GENERALIZATIONS

1. Severe tunnel damage appears to be inevitable when the tunnel is crossed by a fault or fault fissure which slips during the earthquake.

2. In tunnels away from fault breaks, severe damage may be done by shaking to linings and portals and to the surrounding rock, for tunnels in the epicentral region of strong earthquakes, where construction is of marginal quality. Substantial reinforced-concrete lining has proved superior to plain concrete, masonry, brick, and timber in this regard.

3. Tunnels outside the epicentral region, and well-constructed tunnels in this region but away from fault breaks, can be expected to suffer little or no damage in strong earthquakes.

4. While it would seem reasonable that competence of the surrounding rock would reduce the likelihood of damage due to shaking, inadequate comparative evidence is available on this point.

5. Within the usual range of destructive earthquake periods, intensity of shaking below ground is less severe than on the surface.

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TABLE 1. CENTRAL CALIFORNIA 1906

TUNNEL	DESCRIPTION	LINING	DAMAGE	GEOLOGY	DEPTH OF COVER
Wrights No. 1, Santa Cruz Mts., Calif.	SPRR Narrow Gage 6225' long, single track. 13' wide. Reconstructed.	Timber	Overhead caving, crushed timbers, upheaved rails. Broken ties. Offset 4-1/2'. At surface: house split in two, other off foundation.	Sandstone and Jasper, Franciscan age. San Andreas fault crosses 400' from north portal.	702'
Wrights No. 2 near Glenwood	5720'. Reconstructed.	Timber	Broken timbers. Roof caved. Damage from shaking only.	Not cut by fault.	678'

TABLE 2. TOKYO AREA 1923

TUNNEL	DESCRIPTION	LINING	DAMAGE	GEOLOGY	DEPTH OF COVER
Terao	Yokohama Line		Cracked brick portal.		
Hichigama	Yokosuka Line		Landslides at entrance.		
Taura	Yokosuka Line		Interior clear. Landslides at entrance.	Loose surface rock.	50'
Numama	Yokosuka Line		Cracked brick portal.		
Nokogiri-Yama	Honjo Line	Concrete	Walls fractured slightly. Some concrete sloughed off.		
Kanome-Yama	Atami Line		Entrances buried. Masonry portal.	Boulders in slide.	
Ajo	Atami Line		Landslides at entrances damaged masonry portal.		

TABLE 2. TOKYO AREA 1923 (Cont.)

TUNNEL	DESCRIPTION	LINING	DAMAGE	GEOLOGY	DEPTH OF COVER
Ippamatzu	Atami Line	Masonry	Cracks in masonry portals. Masonry dislodged near floor in interior.		
Nagoye (?)	Atami Line		Interior cracked.		100'
Komine	Atami Line	Rein-forced Concrete	Destroyed; RC blocks tilted; ceiling slabs caved in. Formed section cracked.		5-20'
Fudu-San	Atami Line		Cracked masonry portal. Clear in-terior.	Thin, loose material on hillsides.	60'
Meno-Kamiama	Atami Line	Masonry	Partial collapse.	Loose rock.	55'
Yonegami-Yama	Atami Line	Masonry	Minor interior masonry damage. Cracks near portal.		165'

TABLE 2. TOKYO AREA 1923 (Cont.)

TUNNEL	DESCRIPTION	LINING	DAMAGE	GEOLOGY	DEPTH OF COVER
Yonegami-Yama No. 2 or Shimomaki-Yama	Atami Line	Masonry	Portals closed by slides. Deformed interior masonry.		95'
Happon-Matzu	Atami Line		Badly cracked interior. Buried by slides.	Loose material on steep slope.	65'
Nagasaha-Yama	Atami Line	Brick and Concrete	Some interior fractures in brick and concrete.		300'
Hakone No. 1	Tokaido Main Line		Interior cracked		200'
Hakone No. 2	Tokaido Main Line		Undamaged		
Hakone No. 3	Tokaido Main Line Twin Tunnels		Interior cracks, ceiling collapse near portal. Masonry portal damaged.		150'

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TABLE 2. TOKYO AREA 1923 (Cont.)

TUNNEL	DESCRIPTION	LINING	DAMAGE	GEOLOGY	DEPTH OF COVER
Hakone No. 4	Tokaido Main Line		Collapse of loose material. Entrance almost completely buried.		160'
Hakone No. 7	Tokaido Main Line Twin Tunnels		Buried entrance. Landslides. Interior collapse.	Fissured faulted, weathered rock.	100'
Yose	Chuo Main Line		Shallow portions collapsed. Daylighted.	Soft, fine grain rock.	65'
Doki	Sobo Line	Brick	Very shallow, collapsed.		

TABLE 2. TOKYO AREA 1923 (Cont.)

TUNNEL	DESCRIPTION	LINING	DAMAGE	GEOLOGY	DEPTH OF COVER
Namuya	Hojo Line, Boso Peninsula	Brick and Concrete	Cave-in. Cracks with 10" displacement. Landslide.		250'
Mineoka-Yama	Ampo Line, Boso Peninsula (?)	Masonry	Cracks and bulges in masonry from earth pressure.	Some basalt. Geologic Map shows deformed and destroyed rock.	

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TABLE 3. KERN COUNTY 1952

TUNNEL	DESCRIPTION	LINING	DAMAGE	GEOLOGY	DEPTH OF COVER
S.P. No. 3	700', daylighted to 494'.		Faulted, wrecked.	Decomposed diorite. Fracture not noted on surface.	150' ±
S.P. No. 4	334', daylighted and bypassed.		Wrecked, fault cut.	Decomposed diorite. Surface cracks.	125' ±
S.P. No. 5	1170' rebuilt		Fault cut, wrecked.	Decomposed diorite. Cracks to surface. Glory holes.	225' ±
S.P. No. 6	361', daylighted.		Fractured.	Decomposed diorite. No surface cracks.	50' ±

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FIGURES

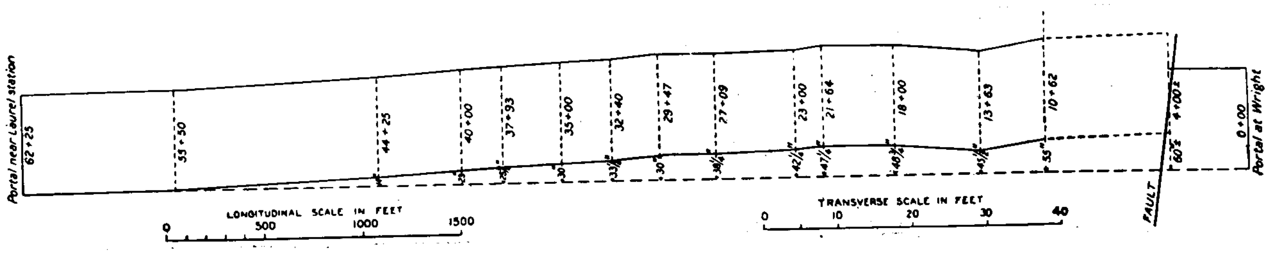

Fig. 1

寫眞第三百二十八

國有鐵道　熱海線根府川眞鶴間國府津起點 9哩 55 鎖 0 節長坂山隧道坑内の被害（國府津方を望む）

Fig. 2

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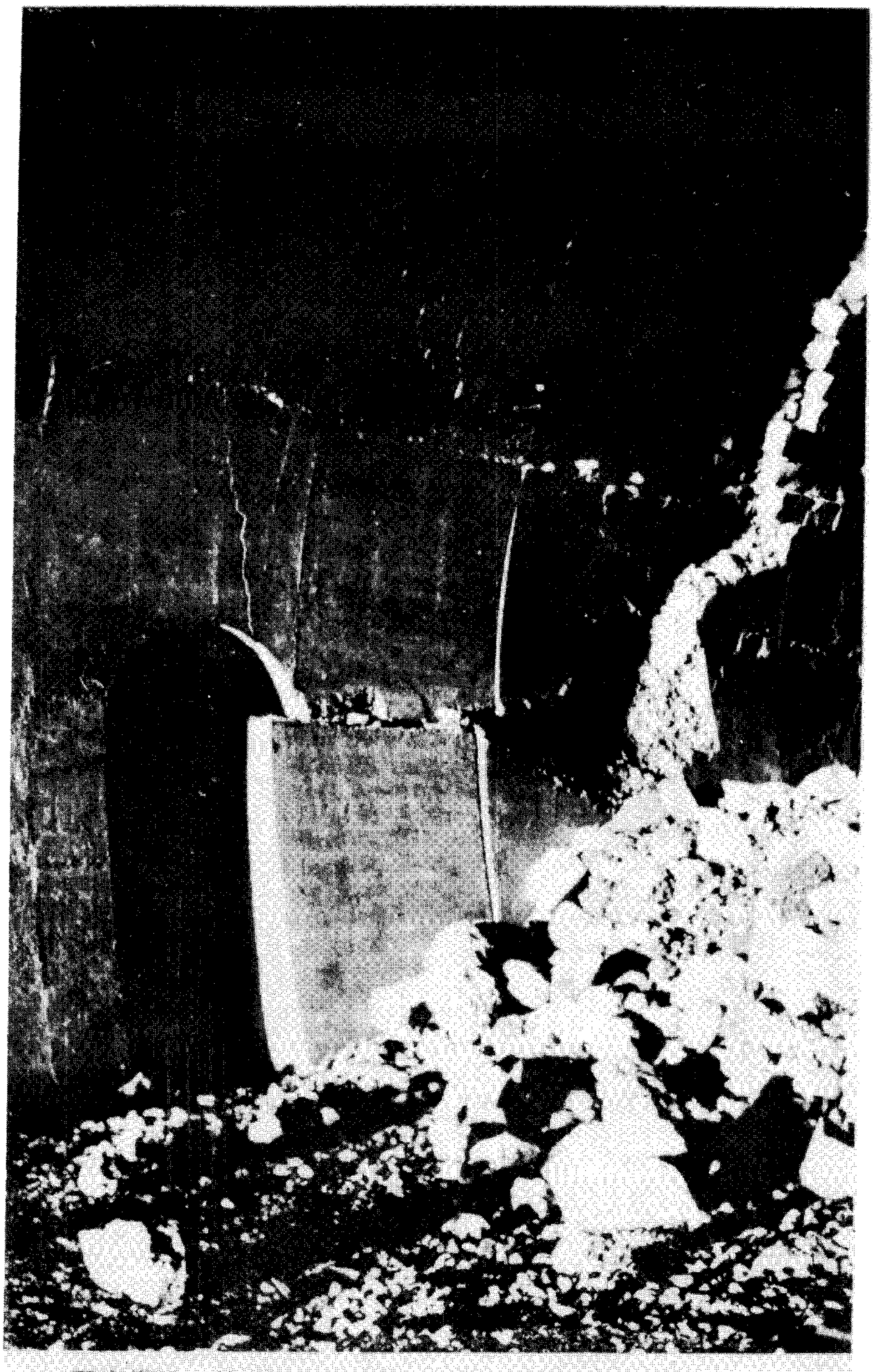
寫眞第三百三十



國有鐵道 熱海線根府川眞鶴間國府津起點9哩55鎖の附近坂山隧道熱海方坑門より約300呎に於ける右側(山側)側壁混凝土の龜裂

Fig. 3

(鐵 道)　附圖第九十三　國有鐵道 北條線南無谷隧道全般圖 Namuya

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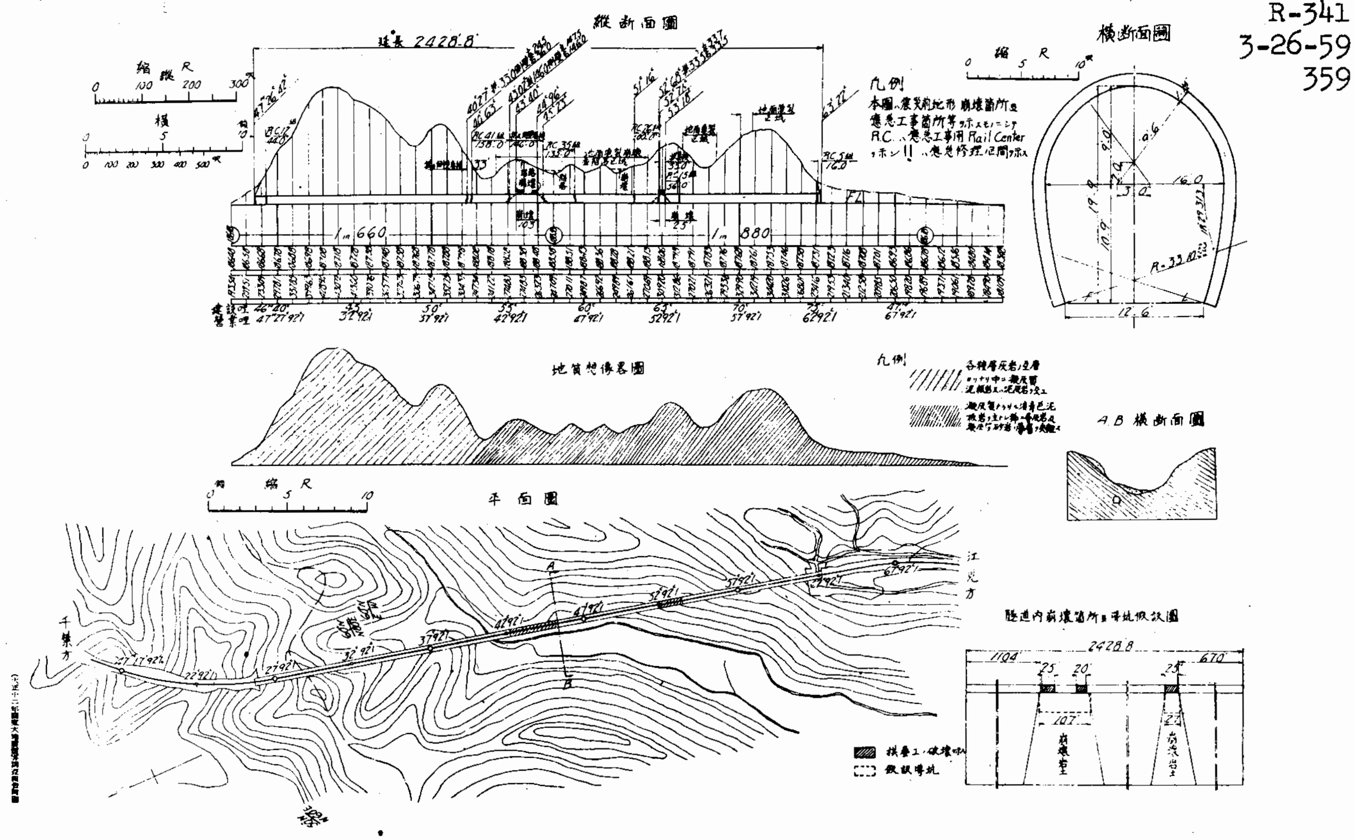

Fig. 4

(鐵 道)　附圖第九十四　國有鐵道 北條線南無谷隧道龜裂圖 Namuya

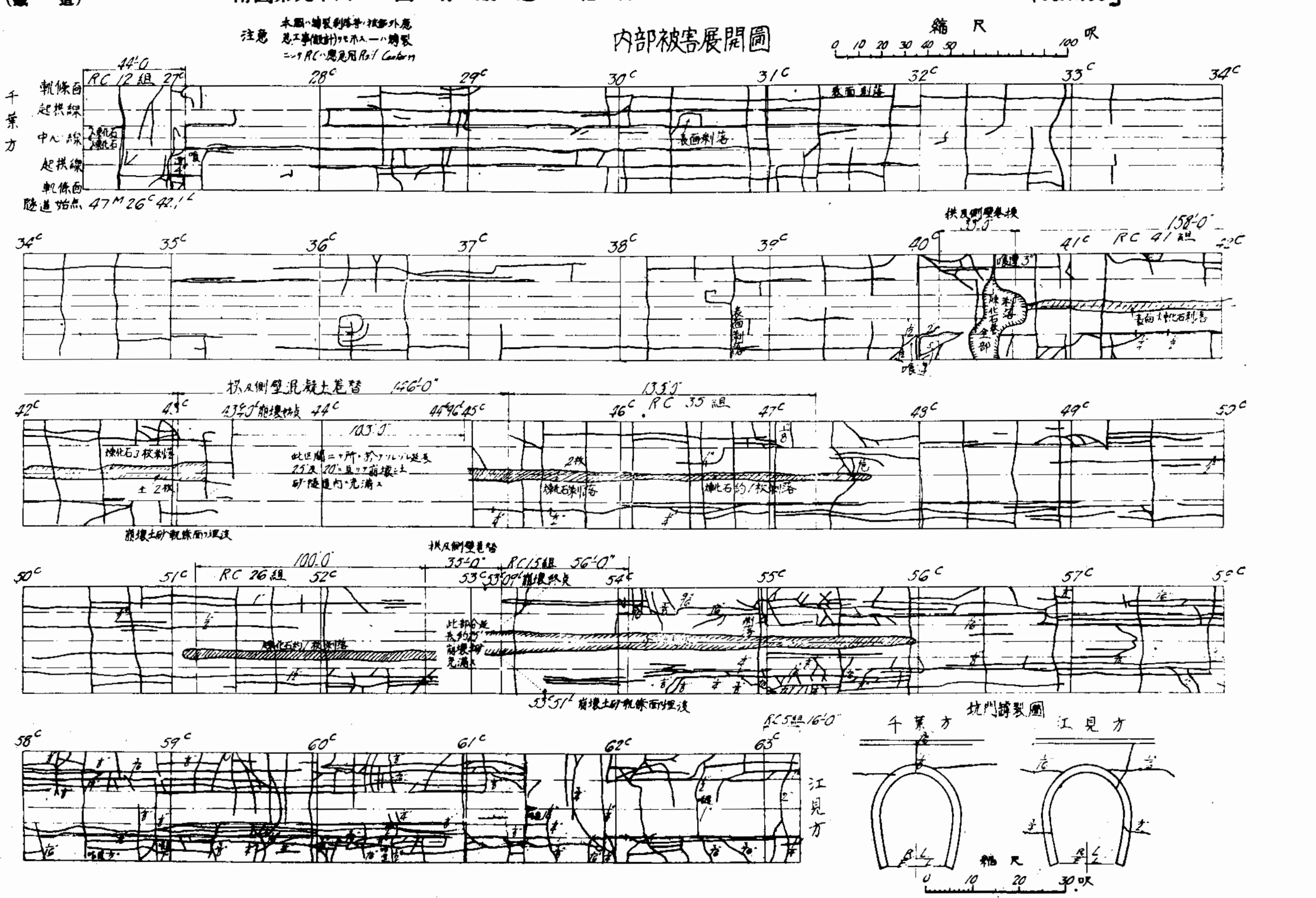

Fig. 5

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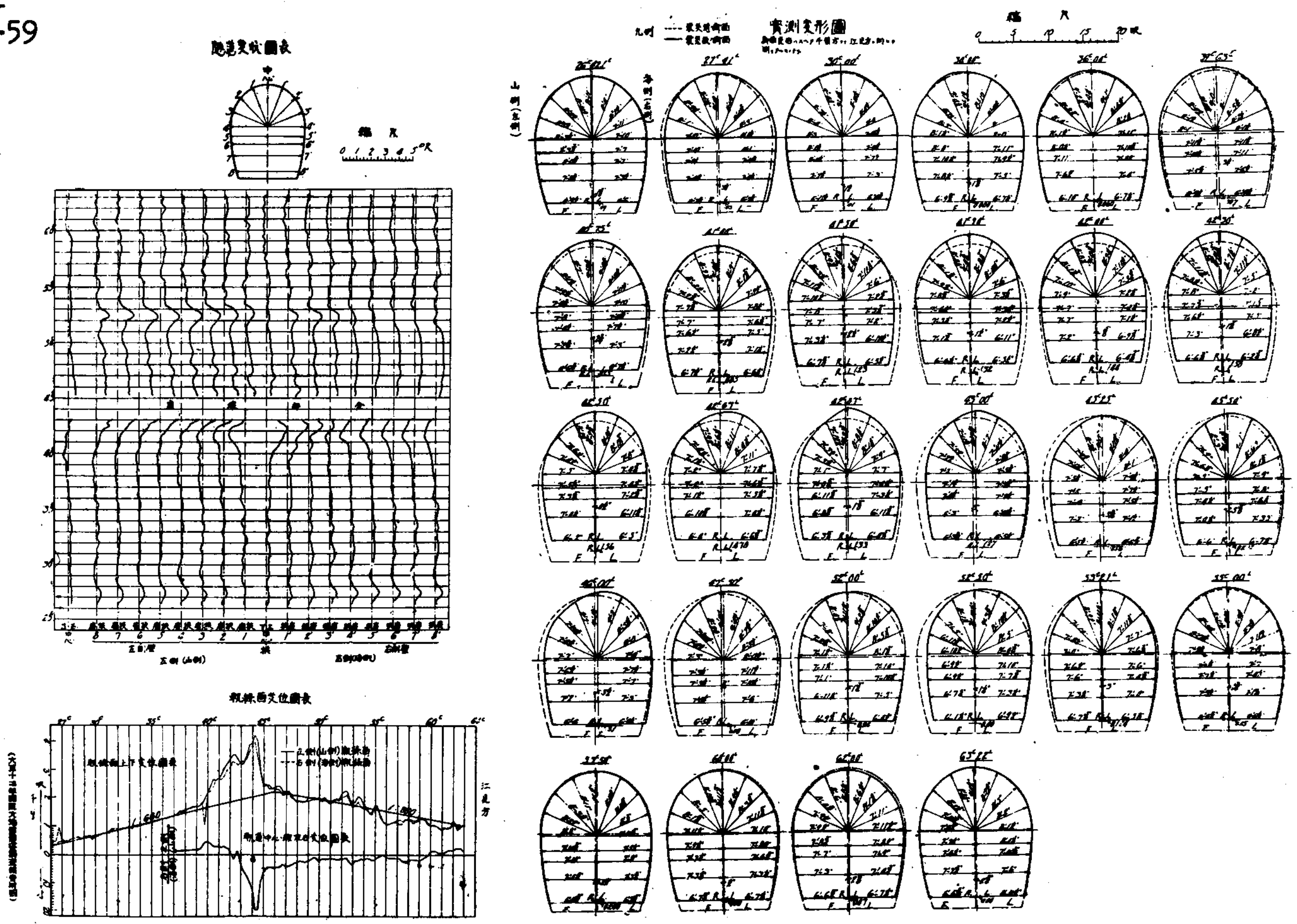
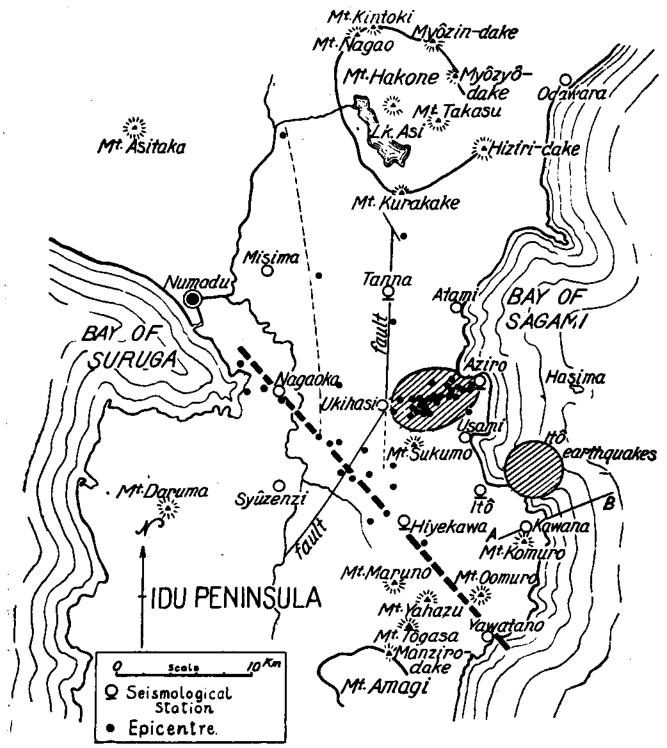
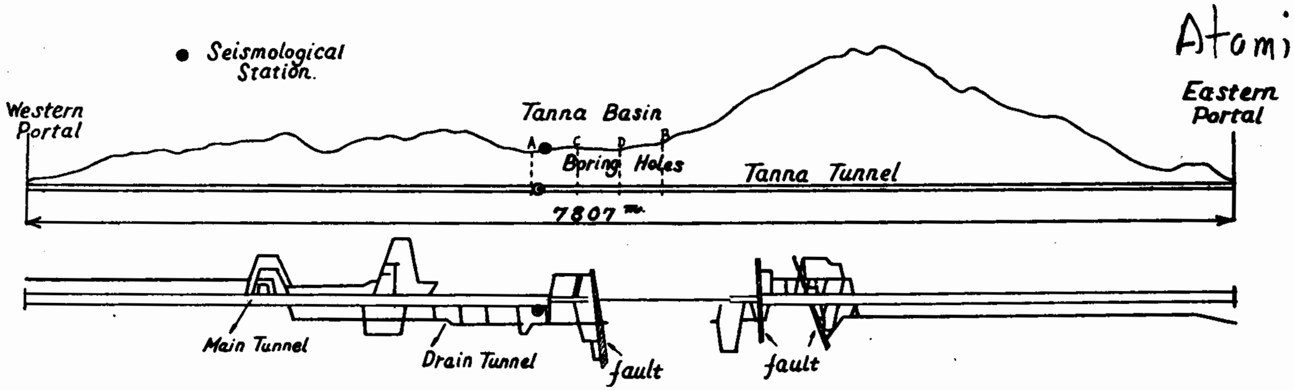


Fig. 6



Distribution of the after-shocks of the Idu earthquake.

Fig. 7



Tanna Basin and the Tanna Tunnel.

Fig. 8

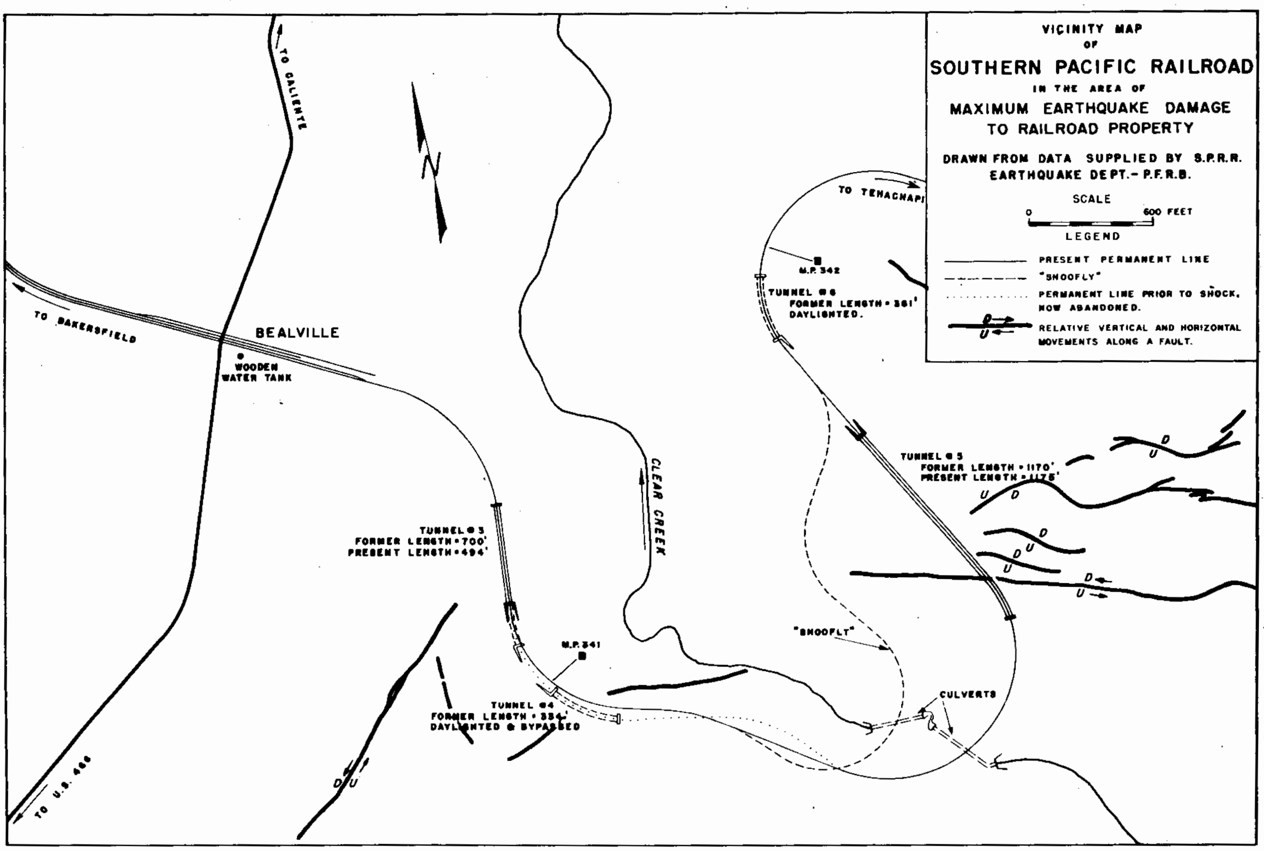


Fig. 9

Fig. 10

Fig. 11

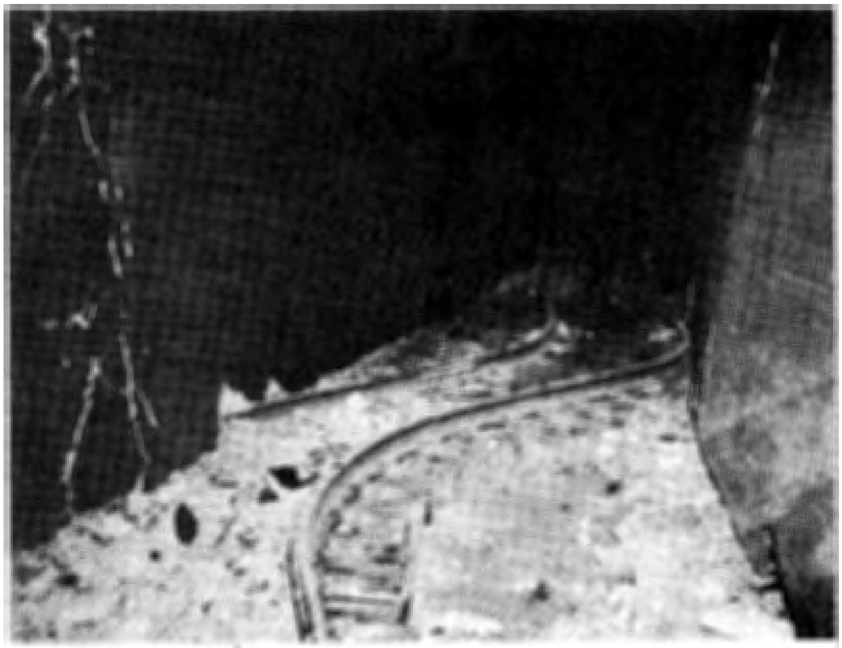


Fig. 12

Fig. 13

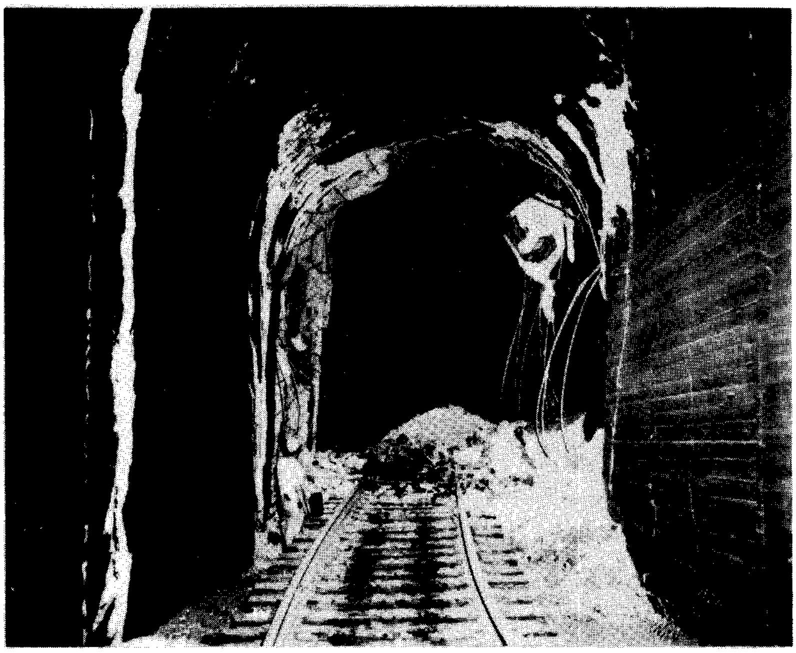


Fig. 15

Fig. 16

REMARKS ON ANALYTICAL METHODS FOR PROTECTIVE STRUCTURAL DESIGN

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INTRODUCTION

The purpose of this paper is to discuss the general nature of the problem of blast-resistant design in terms of the analytical methods that might be used. I am concerned with what Dr. Fritz Zwicky calls the "morphology of the problem," its general "shape" and character, rather than with the details of particular analytical methods.

There are some very good reasons for being interested in the general type of problem rather than in the individual details. One of the most important is the fact that the problem is not clearcut and definite--the size, aiming point, and effectiveness of the weapons we design against are not known, and even if they were, the precise magnitude of the loading produced on a given structure cannot be determined accurately. Moreover, the precise resistance characteristics of a given structure are not known in advance. The difficult part of the design is really the selection of one of several alternatives, and the rough proportioning to decide which of these alternatives should be pursued further. Too often little attention is paid to the preliminary design because so much attention is felt to be needed in so-called "exact" analyses.

Let me be more explicit about the relation between analysis and design. For a given structure with known properties and a definite loading or series of loadings, the analysis of the structural response is essentially a simple problem in mechanics, although sometimes tedious and lengthy. One may often, but not always, proceed to a satisfactory design by modifying the proportions of the structure based on the results of an analysis, and then reanalyzing the structure (for a modified loading, if necessary), until a structure is obtained for which the response is adjudged tolerable. But "design" is basically the choice of the first structure to analyze! And if this is cleverly and appropriately done, the analysis may even be unnecessary, or is done only for the sake of completing the record.

Actually, this sort of design should be done for several different types of structures, even if very roughly, in order that the relative costs and difficulties of construction can quickly be compared, and the proper configuration decided upon.

In view of the nature of the problem and because of the magnitude of the uncertainties associated with it, high precision is not required. If we relax our requirements of precision we may often achieve a simple and rapid estimate of the required structural strength to withstand the given environmental conditions.

In the quick survey which follows of the general relations governing dynamic structural response, attention has been focused on idealized and conventionalized characterizations of the problem. This should not be taken to imply that other factors are unimportant. Construction practices, the fabrication of joints and connections, and the multitude of details that constitute good practice in the building of structures of steel, concrete, earth or other materials, are extremely important and must not be overlooked. Time does not permit further consideration of these matters, however.

In what follows, attention is given briefly to loading, to structural resistance, and to dynamic response relations. Finally some general conclusions are drawn as to the most significant design parameters. Because of the nature of the design problem it is desirable to concentrate on the structural parameters that are most directly useful and most conveniently estimated in advance. For this reason the discussion is concerned with the following structural parameters:

1. The effective "resistance" of the structure, measured in terms of a pseudo-static pressure applied in the same general manner as the blast forces.

2. The effective period of vibration of the structure when it oscillates in a mode corresponding most nearly to its shape as it approaches failure under blast conditions.

3. The ductility factor, or the ratio of the maximum desirable deflection to the deflection at first <u>general</u> yielding.

The corresponding loading paramaters are:

(a) The peak applied dynamic pressure, measured in the same units and acting on the same area as the effective resistance.

(b) The effective duration of the load pulse, considered as a triangular load-time curve. More general cases of loading can, however, be considered.

The aim of the discussion is to relate the required resistance to the required peak loading, in terms of the ductility factor and the ratio of the duration of the loading to the effective period of vibration. Although this gives a quantity which is a measure of the required structural strength under pseudo-static conditions, it is determined by the dynamic nature of the problem and it does not involve any really inadmissible simplification of the actual dynamic situation.

It turns out for most cases where high peak loadings are considered, say even more than 30 psi, for megaton-range weapons, that the required resistance is not appreciably different from the peak loading. There are some notable exceptions, however, where extremely large flexibility or deformability is built into the structure, when the required resistance can be considerably smaller. For such cases, which are characterized by a relatively short duration of loading compared with the effective period of vibration (of less than 0.3 to 0.5) it is most convenient to concentrate on the energy absorption of the structure rather than on its strength. With no more than this very rough guide it is often possible to design a structure satisfactorily ab initio, without having to change its proportions materially after a more detailed analysis!

LOADING

The type of overpressure time curve developed in a blast is shown in Fig. 1 by the solid curve labelled p_s. This curve can conveniently be replaced by a corresponding linear curve varying from a maximum pressure p_{so} to 0 in a time t_s slightly less than t_o. For most structures it is convenient and not inaccurate to neglect the negative phase of the blast beyond t_o. The dynamic overpressure or "drag" pressure is shown in the same curve in relative proportions for a relatively small strength shock by the curve labelled p_d. It also can be approximated by a triangular pulse having a duration somewhat less than the pulse length for the sideon overpressure.

When the blast engulfs a rectangular structure above ground, it is diffracted around and in part reflected from the surfaces of the structure.

It also subjects the structure to drag forces. The pressures exerted on a structure are dependent on the shape of the structure. As a rough measure of the pressures, there are shown in Figs. 2 and 3 the net pressures on the front and the rear, respectively, of a rectangular building. The dimension S in these figures is the shortest distance from the center of the bottom of the front face to the nearest point on the building where the pressures can flow away from this point. The quantity U in the figure is the shock velocity. The net force exerted on the building is the difference between these two curves, and can be roughly approximated by a triangle. The force on the front wall of the structure can also be approximated by a triangle which would be one having a peak value p_r, and a duration which would correspond to an extension of the upper line in Fig. 2 down to the axis. If the period of the element that we are considering is relatively short compared with the time, 3S/U, the "clearing time," then this single initial triangle is an adequate representation of the load. If the period of the element is long, then we must give consideration to the latter part of the load. For a column or a small element the little spike from zero to 3S/U is so short compared with the rest of the curve that we can neglect it and consider the column or other element loaded entirely by drag forces, which correspond to a linear decay from the peak drag force to zero in a time corresponding to the effective drag duration. In either case, for an above-ground structure it is adequate to use a design loading which corresponds to a triangle, as shown in Fig. 4. For a diffraction-type building the triangle starts with the reflected pressure p_r and drops to zero in a time t_1 which corresponds to somewhat longer than the clearing time and is very roughly given by the relationship:

$$t_1 = \frac{L + 3S}{U}$$

For a drag-type building a similar triangle may be used as a loading varying from a peak intensity of pressure p_{do}, the initial drag pressure multiplied by the drag coefficient, and varying to zero in an effective time t_d which is the effective duration of the drag force.

A more complete discussion of the loading on rectangular structures and other shapes of structures is given in Refs. 1, 2 and 3.

For underground structures the peak pressure on relatively shallow buried structures may be considered the peak sideon overpressure, and the duration can be taken as the effective triangular duration, t_s, as in Fig. 1. For arches and domes either above ground or underground the loading is more complicated. A recommended design procedure for such structures is given in Ref. 4.

STRUCTURAL RESISTANCE

The relation between load and deformation, or between resistance and deflection of a structure or a structural element, may take any of a number of forms. It is convenient to define the structural resistance r in the same terms and in the same units as the external loading p. The relationship between r and the deflection x may take the form shown in Fig. 5(a). Here a curve is shown for a relatively brittle structure which fails before much plastic deformation has developed, and for a ductile structure which reaches a sort of general yielding.

Conventionalized resistance deflection curves are shown in Fig. 5(b) for a structure which has an increase in resistance after yielding, or a "work hardening" structure; an elasto-plastic structure which has a uniform plastic resistance after yielding; and two unstable structures which have a decreasing resitance after yielding begins. One of these actually decays in resistance to zero value. Such a structure might have a heavy vertical mass in which the deflection produces additional moment due to the eccentricity of the vertical mass and essentially destroys the resistance of the structure to carry further lateral loading.

A conventionalized resistance curve for a fixed end beam is shown in Fig. 5 (c), where the initial part of the curve corresponds to the situation up to yielding at the ends, the next part corresponds to the development of a larger moment at the middle with a constant moment at the ends equal to the yield moment, and the horizontal part of the curve corresponds to the limit loading of the structure.

Figure 6 shows a more general resistance displacement curve for an actual structure and an effective elasto-plastic relationship which approximates the actual curve. In general, almost any resistance curve can be approximated by an elasto-plastic curve consisting of an initial straight line, not necessarily the same as the initial straight line part of the actual curve, and a horizontal second part which preserves the total area under the given curve and also the area at or near the yield value for the approximating curve. The elasto-plastic curve is characterized by three parameters; namely the yield resistance q_y, the yield displacement x_y, and the maximum displacement x_m which is equal to the quantity μx_y where μ is the ductility factor for the structure. It should be noted that in the replacement curve the yield deflection is not the actual value at which yielding begins, but a sort of effective or equivalent value.

The yield resistance for most materials is increased because of the speed of loading effect, and may generally be taken for steel and reinforced concrete about 25 per cent higher than the normal static value or certainly 25 per cent higher than the minimum specified static yield value.

The ductility factor for various structures ranges from slightly greater than 1 for brittle structures to values of the order of 20 to 30 for very ductile structures. In most cases, however, for long duration loading compared with the period of vibration of the structure, the precise value of the ductility factor does not matter so long as it is in the range of 3 to 5 or greater.

It is possible to plot from a curve such as Fig. 6 or for any actual resistance displacement curve a diagram which shows the energy absorbed by the structure against the displacement of the structure or structural element. For the initial elastic part the energy curve is a parabola and and it is concave upward so long as the resistance of the structure does not decrease with deflection. The curve rises, of course, but tends to be concave downward when the resitance is decaying, or in the unstable range of resistance. Such a curve is shown in Fig. 7.

ENERGY RELATIONS

There are two cases where an exact relationship can be derived for the peak dynamic force to be applied to a structure having a given resistance. These two cases correspond to (1) a situation where the dynamic force rises suddenly to a maximum value and remains constant for all deflections of the structure; and (2) the case where all of the force is applied as an impulse before the structure deflects appreciably. The first of these cases corresponds in Fig. 7 to a line such as OB drawn from the origin, having a constant slope corresponding to the maximum dynamic force p_m. The work done on the structure by the dynamic force is, of course, the force p_m multiplied by the deflection, and a straight line through the origin represents this quantity. Where it intersects the curve the absorbed energy is equal to the external energy, and we have the maximum deflection x_m produced in the structure.

In the second instance we can consider that the impulse I from the loading is applied to the structure in such a way that the masses of the structure acquire an instantaneous velocity. The magnitude of the instantaneous velocity at each mass is equal to the impulse divided by the mass. From this one can derive the fact that the kinetic energy of each mass is equal to the square of the impulse divided by twice the mass. By summing these values one can arrive at the total kinetic energy and plot it as a quantity of input energy such as at F in Fig. 7. A straight horizontal line drawn from this point, such as FC, will intersect the curve at an absorbed energy equal to this initial kinetic energy and will therefore give the maximum displacement x_m' produced by the impulsive loading. The maximum deflection can also be obtained for the situation where a long duration load is combined with an initial impulse. Here one draws a line of constant slope equal to the magnitude of the long duration loading from a point such as G, that corresponds to the energy of the initial impulse.

There are some minor points to be considered with this kind of graphical solution. The most important of these is the fact that in an unstable resistance curve, it is possible that some particular point on the curve such as E cannot be reached by a line drawn from a particular point on the vertical axis such as G without such a line passing through some intermediate point of the energy displacement curve. This situation merely means that the curve from the initial starting point tangent to the energy absorption curve, as at D, is the one that should be used, as all deflections beyond this point will be produced by a loading infinitesimally greater that the one corresponding to a tangent intersection.

Relationships are given in Ref. 1 for analytical relationships which define the same situation as is described in the curve in Fig. 7, for the case of an elasto-plastic resistance. In this reference a general empirical result is developed which is applicable to other than infinitesimally short or infinitely long durations of loading also.

RESPONSE CHART

A plot of the dynamic relations obtained by exact solutions for a triangular pulse loading on an elasto-plastic system is shown in Fig. 8, and more accurately in Fig. 9. In Fig. 8 the plot is in terms of the ratio of the yield resistance to the applied peak force, and in Fig. 9 the inverse of this is plotted, namely the maximum pressure on the structure divided by the yield resistance. The curves of Fig. 9 give relationships among the following four quantities:

x_m/x_y or ductility factor μ, which is shown on the scale on the left hand side of the figure.

t_d/T or the ratio of the duration of a triangular load pulse to the effective period of the structure, shown as the abscissa.

p_m/q_y, which is the ratio of the peak pressure applied to the structure to the effective yield point resistance, measured in terms of pressure. Lines of equal values for this ratio are shown by the lines which slope generally diagonally up and to the right on the figure.

t_m/T, or the ratio of the time at which maximum deflection is reached to the effective natural period of vibration. These are shown by the dashed lines sloping generally down and to the right on the figure.

In general, for relatively short durations, or in the region where the sloping lines have a nearly constant slope equal to that at the left hand side of the figure, impulse governs the response. It can be seen that this is the case for durations of loading less than about one-third of the period of vibration for elastic structures, and extending to longer durations, up to two or three times the period of vibration, for structures in which a large amount of plastic energy is absorbed. In this region of the chart energy absorption governs the structural response.

On the right hand side of the figure, where the lines of equal ratios of resistance to peak loading have a horizontal tangent, the loading is effectively very long, and the required resistance of the structure is very nearly equal to the peak loading. It can be seen that for ductility ratios greater than about 2 and less than 20, if the duration is more than about five times the period of vibration, the required yield resistance of the structure is approximately equal to the peak dynamic peak pressure applied to the structure, within less than a 25 per cent error.

For more complex loading curves it is possible to use the same chart with a reasonably accurate degree of approximation by dividing the loading into a series of triangular components as in Fig. 10 and using the relationship:

$$\sum \frac{p_n/q_y}{F_n} = 1$$

In this equation F_n corresponds to the ratio p_n/q_y if the particular loading p_n with its particular duration were the only force acting on the structure. A more detailed explanation of this technique is given in Ref. 5.

Where more accurate calculations are desirable, numerical techniques are available for making such calculations. A summary and description of a group of techniques that may be used either for hand computation or with a high-speed digital computer is given in Ref. 6.

REBOUND

When any structure is loaded and reaches a maximum deflection, it has energy stored in it and tends to deflect backward or in the opposite direction. This tendency exists even in the case when there is still some forward loading acting on the structure at the time it reaches its maximum deflection. In general the rebound is elastic, although in the case of a reinforced concrete structure, if the rebound is very large and sufficient steel in the reverse direction is not provided, there may be an inelastic part of the rebound. Consideration must be given to this situation particularly in design of reinforced concrete structures. Where the structure is in the impulsive loading region, the rebound resistance must be equal to the forward resistance in general, but for the long duration region of the charts shown in Figs. 8 and 9, the rebound resistance may be considerably less. In no case should it be less than about 25 per cent of the forward resistance.

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DISCUSSION

MR. ROSS CASH (Moffatt & Nichol, Engineers, Long Beach, California): You mentioned that the chart you showed in one of your slides was available. Where is that?

PROFESSOR NEWMARK: The chart is available in a paper that I published in Transactions of the American Society of Civil Engineers called "An Engineering Approach to Blast-resistant Design" and, in more detailed form, in a number of technical reports that have been issued in various places. One of the latest is in a guide issued by the Secretary of Defense.

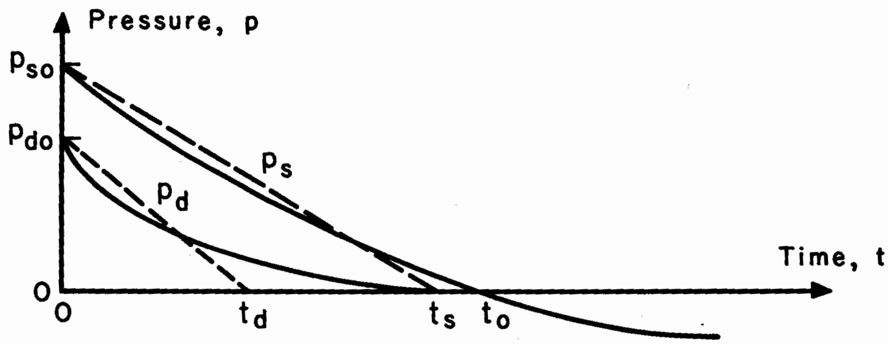

Fig. 1 — Side-on overpressure and drag pressure

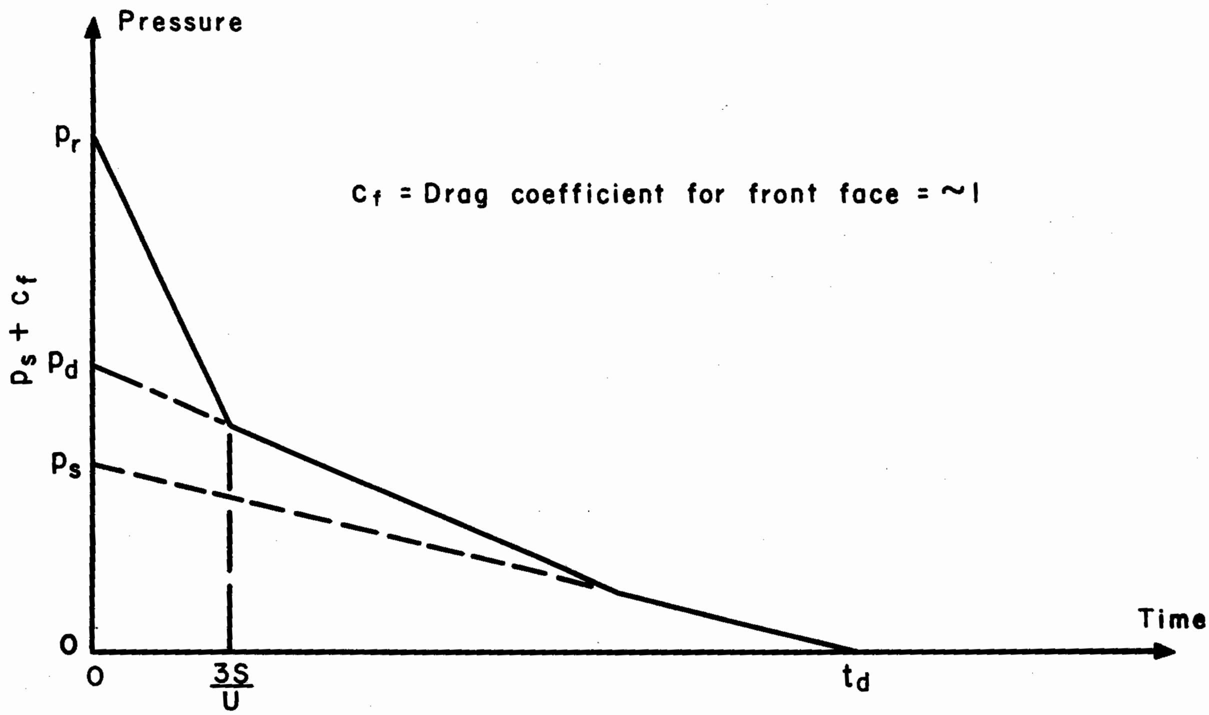

Fig. 2 — Net pressures on front of rectangular building

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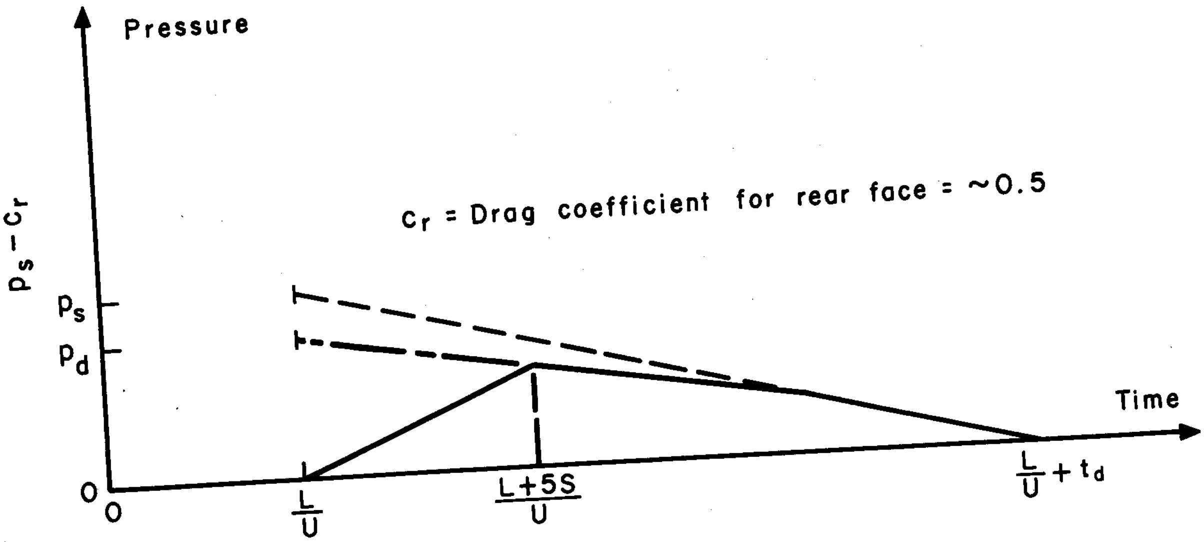

Fig. 3 — Net pressures on rear of rectangular building

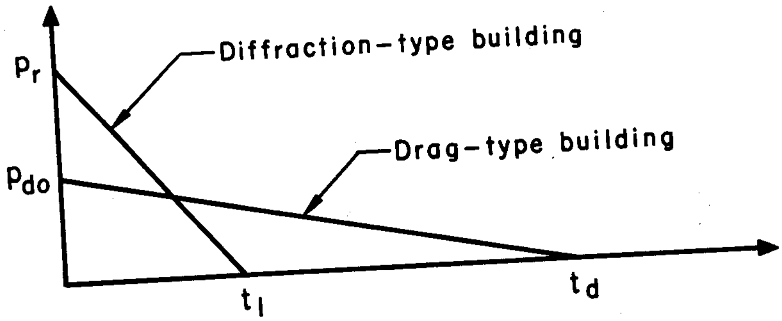

Fig. 4 — Recommended design loadings

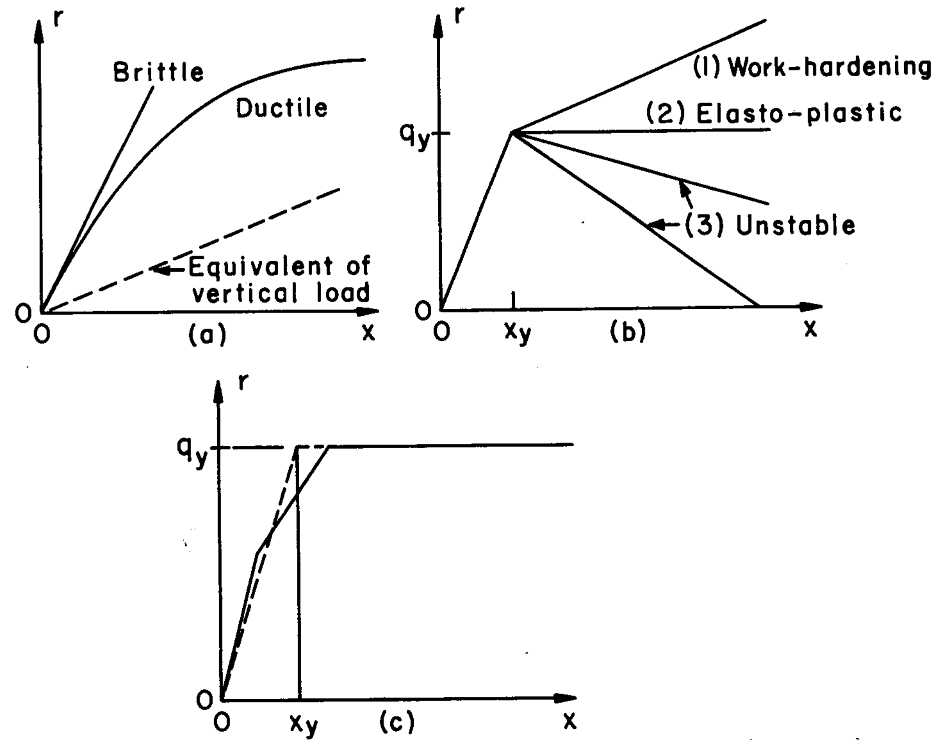

Fig. 5 — Typical resistance — deflection relations

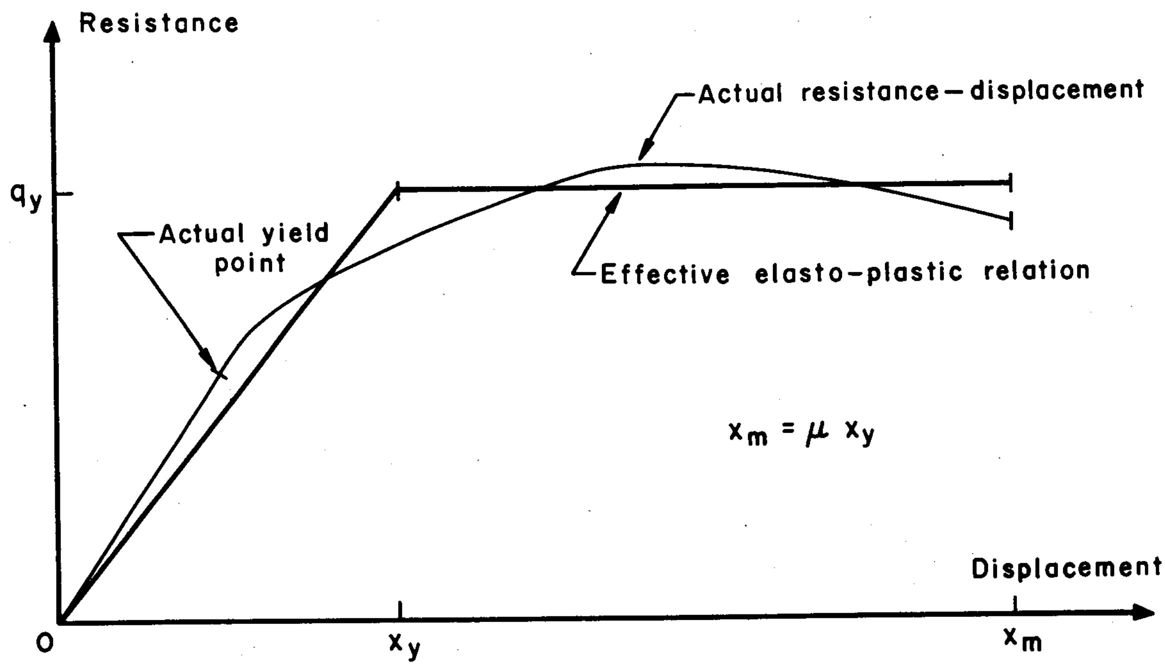

Fig. 6 — Resistance — displacement relationship

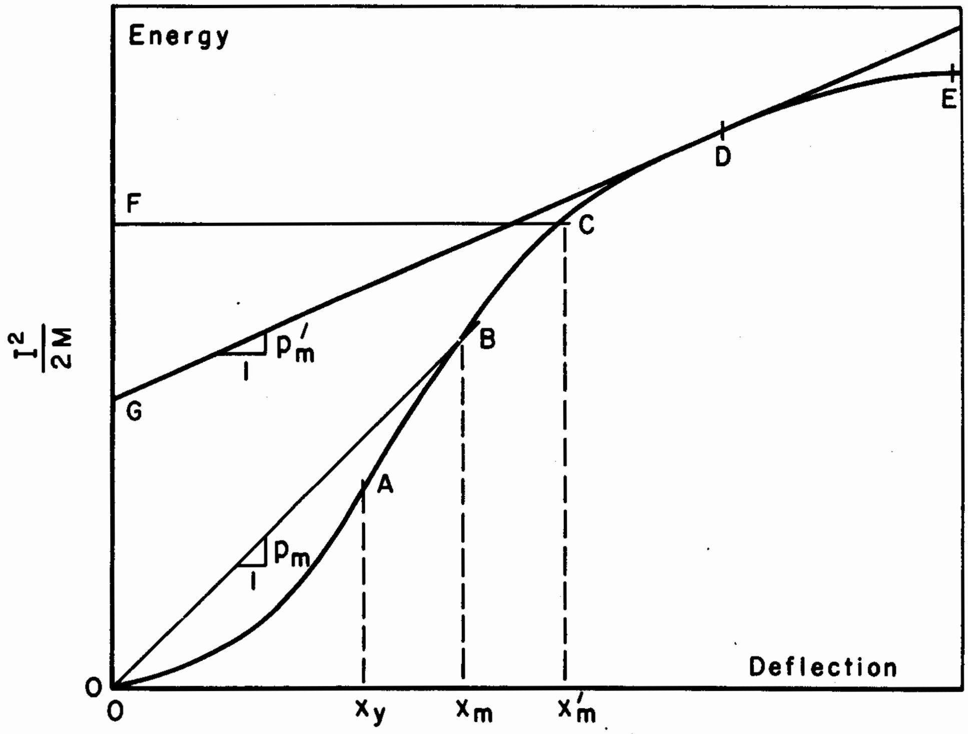

Fig. 7 — Energy absorbed versus deflection

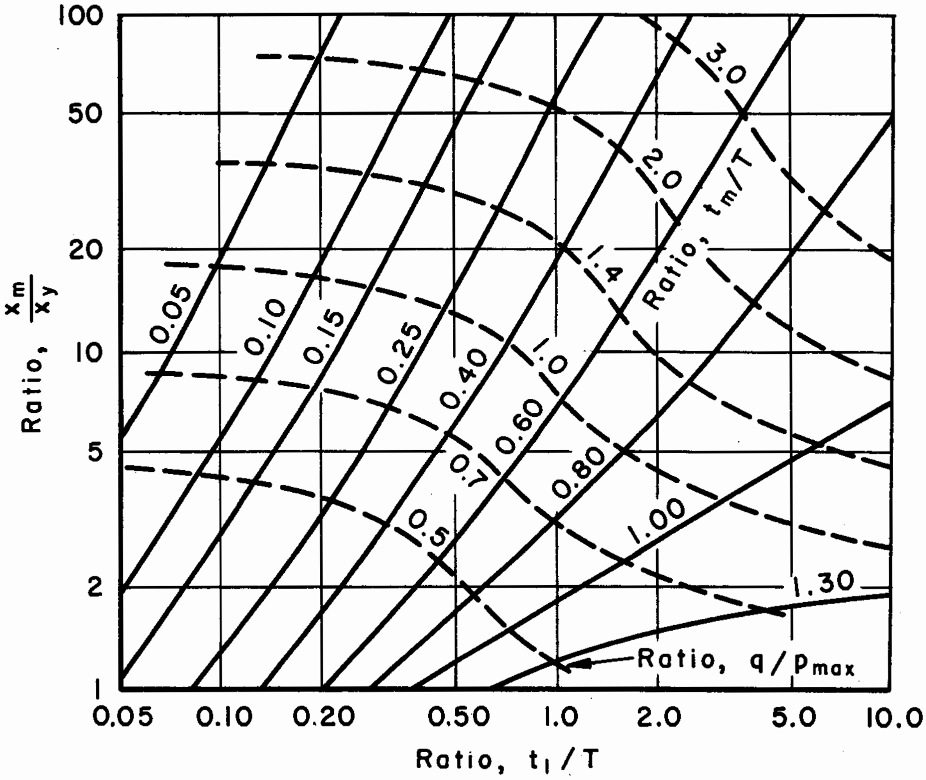

Fig. 8 — Response, pressure, and resistance relations for triangular pulse loading

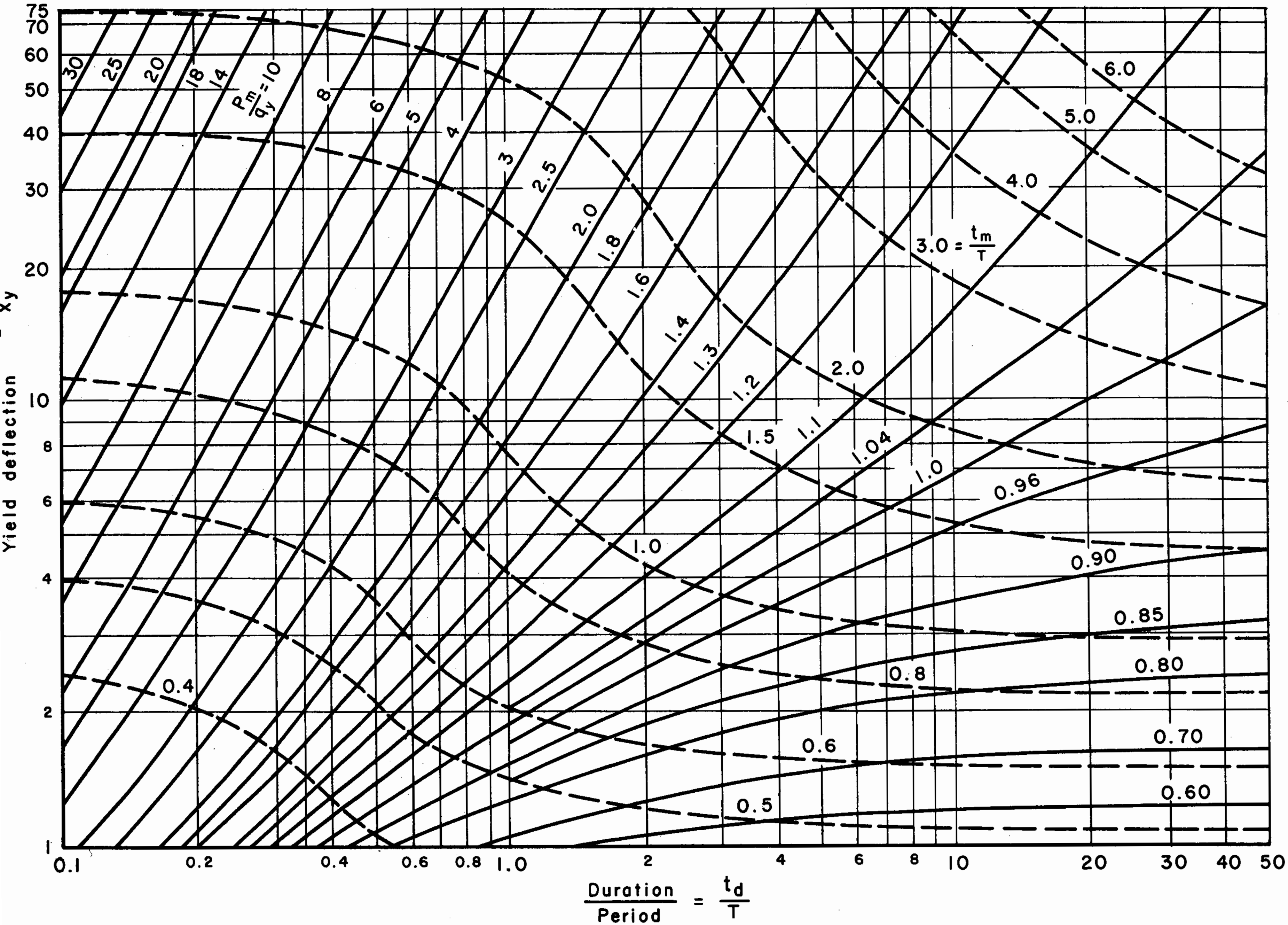

Fig. 9 — Design chart for initial peak triangular force pulse on elasto-plastic system

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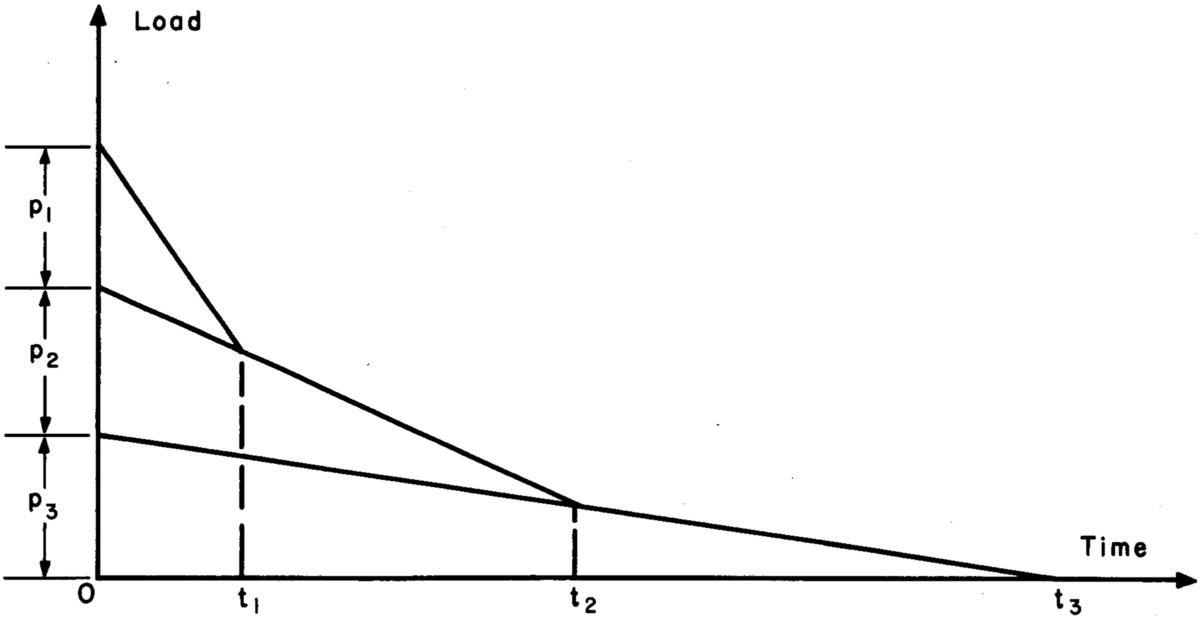

Fig. 10 — Typical load-time relation considered

BLAST VULNERABILITY OF DEEP UNDERGROUND FACILITIES AS AFFECTED BY ACCESS AND VENTILATION OPENINGS

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INTRODUCTION

If it were possible to build deep underground installations without openings to the surface, the hardness of the facilities would be controlled only by the integrity of the underground rock chamber and the ability of the interior construction to withstand the effects of ground shock. Once openings are provided, the possibility of the entrance of blast pressures through the openings into the interior must be considered. The statement is often made that this condition severely limits the ultimate hardness of what may be an otherwise invulnerable facility. I hope to indicate that with careful design these limitations can be overcome, although considerable expense may be involved in some cases.

If the level of protection for design is set at a lower level than the potential hardness of the underground chamber, as located and constructed, retrofitting of air intakes and closures at a later date can be readily accomplished. However, if the potential hardness of the underground chamber is too low, retrofitting is almost impossible. For this reason, planning of deep underground facilities should not be controlled, in terms of hardness, by the limitations of existing closure systems.

Openings may be divided into two broad classifications, those which are open only intermittently to allow the entrance of personnel or equipment, and those which must remain open continuously for ventilation or similar purposes, except during passage of the blast wave.

PERSONNEL AND EQUIPMENT ENTRANCES[1][2]

The hardness problems created by the first group can be readily overcome by dividing all such entrances into a series of pressure locks with electrically interlocked blast-resistant doors on both sides of each pressure lock, as shown in Fig. 1. The interior facility is then continuously closed off from the surface through the personnel and equipment exits and entrances. The blast doors in these passageways can be made nearly as hard as the underground facility itself. In addition, it may be desired to make the passageways themselves "fail safe" at some specific pressure level. In many cases, such as at command posts, the cost of personnel and equipment doors used for pre-shot operation and maintenance can be kept to a minimum by not requiring that they operate after an attack, only that they remain hard and protect the interior from repeated attacks. Personnel access following an attack would be through small-diameter tunnels with blast covers or doors also arranged in pairs in "safety locks" and interlocked. The normal passageway doors and the emergency tunnel doors can be readily protected against thermal and nuclear radiation effects by additional concrete thickness.

Because of the safety locks, the time to open or close the doors does not affect the hardness of the installation and is determined as a function only of operational convenience. The type of door used will be a function of the design requirements. Personnel doors have been designed and tested at peak pressures up to several hundred pounds per square inch. The following types of doors have been tested at the Nevada Test Site: flat and arch steel plate, welded wide-flange sections, vault,[3] and reinforced concrete[4] (see Fig. 2, a, b, c, and d). Any of these, as well as numerous other designs, can provide adequate protection under proper conditions.

Emergency exits may be closed off by domed or dished, built-up or flat steel plates in regions up to the fringes of the fire ball. A typical design, which has been tested at a high pressure level, is shown in Fig. 3. For very high pressures, solid concrete plugs may be used. Depending on the weight of the door construction, mechanical assistance will be required for post-attack opening. This equipment must be positive and simple. Again, speed of operation is of minor importance, unless it is anticipated that these exits may be used for personnel making exterior repairs during the attack period.

Once structural adequacy is assured, the problems in the design of large- and heavy-equipment access doors are primarily concerned with automatic operation. This is within the range of conventional design practice, as illustrated by heavy shielding doors for reactor facilities, or the 300-ton movable roof sections which are currently being constructed for a peacetime auditorium project.[5]

For deep underground facilities which, except for surface openings and passageways, are adequate to survive directly overhead surface bursts, the passageways should all be arranged to fail safe by assuring that they will be blocked by their own collapse at a loading which is less than that required to collapse the doors (see Fig. 4a). Fail-safe baffles may also be built into the tunnels for the same purpose, as shown in Fig. 4b.

Although part of the blast wave will enter the passageway prior to closure of the opening by the debris from the fail-safe tunnel section or baffles, the major part of the wave will be excluded. It then becomes practical to design the tunnel beyond the blocked opening with sufficient length, volume, and baffling to attenuate the blast and limit the pressure acting on the blast doors to a value consistent with the door hardness. Obviously, this approach is economically useful only when the additional door hardening is more expensive than the fail-safe devices and additional tunneling, or when reasonably complete protection is needed. It may be combined with considerations of blast attenuation in tunnels and other ways of attaining these results, which are covered in this symposium in the paper by Dr. H. L. Brode on shock wave attenuation in tunnels.

If, at some future date, it were decided to retrofit the passageways to a higher pressure level, the doors and baffles could be strengthened or replaced. It should be noted that the fail-safe baffles or tunnels are in a time contest or race with the shock wave to reach the unprotected opening to the underground chambers and must come very close to winning the race. This is one case in which overdesign or excess strength can be very dangerous. It can be seen that with the above approach, multiple attacks are not of great significance for openings for intermittent use.

VENTILATION OPENINGS

Let us now consider openings in the second category, those which must remain continuously open except during passage of a blast wave.

For short-duration blast waves, devices such as baffles, mufflers, etc., which restrict the openings and retard the passage of the blast wave, are partially effective when used with plenums of adequate size. For long-duration waves, for such facilities as small personnel shelters where normal non-operational air requirements are small and alerts are of short duration, sand filters may be useful in the attenuation of the pressures. In "Operation Plumbbob"[6] a cubic meter of coarse sand placed over air intakes allowed only a 0.2-psi pressure rise in a 1200-cu ft chamber at a peak incident pressure on the ground of 115 psi. Of course, it must be noted that the duration of the positive phase was only 0.2 sec. With this type of filter, the pressure continues to increase throughout the positive phase. For megaton weapons the pressure rise would be higher.

Where large volumes of air must be handled during normal operation, positive closures are needed which, under normal operating conditions, do not unduly obstruct the air passage; these obstructions would increase the pressure losses in the system, increasing the ventilation power requirements. Various blast valves of this type have been developed during the past ten years. These closures are of two types, blast-actuated (depending on the blast pressure to close it) and remote-actuated.

In principle, a blast-actuated valve can be made completely blast tight by extending the required blast travel path sufficiently to allow the valve to close before the blast reaches the interior, as shown in Fig. 5. In Fig. 5a the blast path to the chamber entrance is increased by increasing the duct length; as an alternate, the expansion chamber may be used to delay the blast. In Fig. 5b the sleeve valve is located so that it is exposed directly to the blast before the blast reaches the valve openings through the ducts. In this way, closure is made before the blast can enter the inlet to the interior facility.

Although the valves which will be discussed are not completely blast tight, it must be remembered that there are different ways to approach the ideal seal and that the ideal way may not necessarily be required in particular cases. However, it does appear that positive sealing is possible if it is required.

The valves which will be discussed do not exhaust all the possibilities. In fact, many new designs are being proposed and studied and new ideas are always needed. Three such valves are shown in Fig. 6. Figure 6a is a ball-check valve[7] in which, under the action of a blast, the suspended ball is driven forward onto the spherical seat, closing off the duct. Figure 6b is a swing-check valve[8] which can be designed to be hand-operated or blast-actuated. Figure 6c shows a sleeve-type valve which is driven down under the action of blast to close off the ports. Another possibility being reviewed is the feather-type valve which may be adapted from commercial practice for certain types of blast valve applications. Most of these valves may be either single- or double-acting. It is important to note that metal valves and moving parts must be protected from thermal effects and all openings protected from debris when considering high pressure levels resulting from an attack with high-yield weapons.

For present valves, closing time under blast conditions and the size of the orifice will determine the volume of the expansion or plenum chamber behind the valve which is necessary to limit the pressure rise to any specified maximum value. The closing time for a poppet valve depends on the area and shape of the disk, the magnitude and distribution of pressures acting on the disk, the mass of moving parts, disk travel distance, and restraining or friction forces. Changes made with the objective of decreasing closing time may adversely affect the strength of the valve or resistance to normal air flow. A short closing time must be obtained by making the valve disk diameter and disk travel as small as possible consistent with the low pressure drop required under normal ventilating conditions.

In addition to providing protection against the positive-phase pressures, the valves should be latched or double-acting for protection against negative-phase pressures where the installation is subject to damage from this effect.

Although field tests of blast valves are described below, caution should be used in interpreting the effectiveness of valves from test results unless exterior pressure-time curves are available. The relationship between the interior and exterior peak pressures obtained from field tests on these valves may be misleading where the blast wave was non-ideal. In tests where this was the case, the valve may have closed long before arrival of the peak pressure and at a pressure which was only a fraction of the peak.

In 1950, at the U.S. Army Chemical Warfare Laboratories, development work was begun in connection with the instrumentation of a CBR shelter for the Corps of Engineers at "Operation Greenhouse"(9) to develop a device which could protect the filter from blast effects. The device, designated the E4 valve (later M1), is shown in Fig. 7, a and b. At the normal air flow rate of 300 cu ft/min a light plate sustained by a spring permitted the air flow to pass around the plate and through the perforations in the bedplate. Under pressure, the light plate seats on the bedplate, sealing off the system to further flow. The E4 is equipped with flanges for interconnection with standard pipe (6-in. blast side, 5-in. protected side). This valve was successfully tested at "Operation Greenhouse" and was one of the earliest developments in this field.

Following "Operation Greenhouse" the work on antiblast closure devices was discontinued by the Chemical Warfare Laboratories. Soon thereafter, the Office of the Chief of Engineers initiated a development program to assess the relative merit of other types of pressure-reducing mechanisms such as rock grills, mufflers, baffles, pipe tees, etc. These projected blast-dampening mechanisms were partially evaluated on the shock tube at Aberdeen Proving Ground and field-tested at Nevada Proving Ground in "Operation Upshot Knothole."(10) It was concluded that a quick-acting closure valve would provide the best blast protection for a ventilation system. The data associated with the E4 valve are shown in Table 1.

Late in 1954, the Chemical Corps was requested to equip two prototype civil defense shelters in "Operation Teapot."(4) The subsequent development work prepared one non-mechanically ventilated shelter and a second mechanically ventilated shelter. These shelters were evaluated at Nevada Proving Ground in early 1955.

In the non-mechanically ventilated shelter, pressure gages both ahead of the filter system and in the shelter proper showed zero pressure readings.

The E4 valve performed well in the mechanically ventilated shelter in preventing damage to the blower motor, filter, elbow and air register, and the anti-back-draft valves. The peak incident pressure was 100 psi.

In 1956, development of the E19 valve(11)(12) was initiated by the Chemical Corps on the request of the Bureau of Yards and Docks. The objective was an antiblast valve having a rated flow of 600 cu ft/min for use with pressurized types of CBR shelters. This valve is shown in Fig. 8, a and b.

Additional design requirements of the Bureau of Yards and Docks were as follows:

1. Mode of actuation: blast actuated

Table 1

BLAST VALVE DESIGN CRITERIA, PHYSICAL DATA AND TEST RESULTS

Valve .... E4 (now M1) (Fig. 7)

Nominal size .... 6 in.

Valve body
  Diameter .... 16-1/2 in.
  Weight .... 330 lb
  Material .... cast steel

Disc
  Diameter .... 10-5/8 in.
  Travel .... 1 in.
  Material .... aluminum

Weight of moving parts .. 1 lb

Seat gasket material .... synthetic rubber

Maximum side-on blast pressure ... 100 psi

Rated air flow .... 300 cu ft/min

Pressure drop at rated flow ...... 1 in. water

Inlet area .... 28.9 sq in.

Open area through valve .... 26.8 sq in.

Ratio valve/connection 0.93

Seat opening .... 18, 1-1/4-in. holes
6, 1-in. holes

Protected side duct area .... 20 sq in.

Field tests
  Date .... March-June 1953
  Location .... Nevada Proving Ground
  Summary of data .. shown below

Incident pressure (psig)	Mounting position	Protected chamber (cu ft)	Closure time (msec)	Maximum pressure in chamber (psig)	Total leakage (cu ft)	Average leakage rate (cu ft/sec)
20.3	vertical	87	unknown	0.3	2	--

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2. Method of re-opening following the blast: manual

3. Design overpressure: 50 psi

4. Rated flow of 600 cu ft/min: less than one-in. water gage

Shock tube tests were conducted on the E19 by Ballistic Research Laboratories at Aberdeen Proving Ground, and a single prototype was evaluated under field conditions in "Operation Plumbbob."(6) The data associated with the E19 valve are shown in Table 2.

The valves shown in Fig. 9 were designed and constructed by Arthur D. Little, Inc., under contract with the Office of Defense and Civilian Mobilization and fabricated in 12, 16, and 24-in. nominal sizes.

Nominal Size (in.)	Rated Air Flow (cu ft/min)	Maximum Pressure Drop (in. water)	Maximum Side-On Blast Pressure (psi)
12	600	1	100
16	1200	1	50
24	2500	1	50

The valves may be equipped with a remote-operated pneumatic cylinder attachment for closing the valve prior to arrival of the blast. Closing may be accomplished by using a photo-electric relay exposed to the fireball for triggering the air cylinder. The air cylinder is designed to hold the valve closed against a negative pressure of 5 psi. The data associated with the ODCM valve, as obtained in "Operation Plumbbob," are shown in Table 3.(6)

The OCE valve (Fig. 10, a and b), which was developed by the Corps of Engineers and has been used in numerous installations, is a remotely operated poppet, rated 5000 cu ft/min maximum capacity with approximately 1-in.

Table 2

BLAST VALVE DESIGN CRITERIA, PHYSICAL DATA AND TEST RESULTS

Valve .... E19R1 (Figs. 8a and 8b)

Nominal size .... 8 in.

Valve w/housing
- Over-all dimensions .. 21-1/4 x 18-5/8 x 17-3/4 in.
- Weight .... 130 lb
- Material .... steel

Louver doors
- Size .... 2, 4 x 15-1/2-in. doors at 30°
- Travel .... 2 in. plus
- Material .... aluminum

Weight of moving parts . 1 lb per door approximately

Seat gasket material ... neoprene sponge rubber

Maximum side-on blast pressure .. 50 psi

Rated air flow .... 600 cu ft/min

Pressure drop at rated flow ..... 0.4 in. water gage

Inlet area .... 50 sq in.

Open area through valve .... 60 sq in.

Ratio valve/connection 1.2

Seat opening .... 105 sq in.

Protected side duct area .... unrestricted

Field test (E19 valve, predecessor to E19R1, with 30° door opening)
- Date .... June 1957; Priscilla Shot of Operation Plumbbob
- Location .... Nevada Proving Ground
- Summary of preliminary test data .. shown below

Incident pressure (psig)	Mounting position	Protected chamber (cu ft)	Closure time (msec)	Maximum pressure in chamber (psig)	Total leakage (cu ft)	Average leakage rate (cu ft/sec)
64	horizontal; 5 ft below grade; fed by 8-in. steel pipe and tee weatherhead	194	--	0.32	4.77	--

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Table 3

BLAST VALVE DESIGN CRITERIA, PHYSICAL DATA AND TEST RESULTS

Valve	ODCM (A.D. Little)(Fig. 9)[(6)]
Nominal size	16 in.
Valve body	
Diameter	23.5 in.
Weight	760 lb
Material	ductile iron
Disc	
Diameter	16 in.
Travel	2.38 in.
Material	aluminum
Weight of moving parts	31 lb
Seat material	ductile iron
Maximum side-on blast pressure	50 psi
Rated air flow	1200 cu ft/min
Maximum pressure drop	1 in. water
Inlet area	120 sq in.
Open area through valve	123 sq in.
Ratio valve/connection	1.03
Diameter of seat opening	13.75 in.
Protected side duct area	182 sq in.

Field tests
Date .... May-October 1957
Location .... Nevada Proving Ground
Summary of data .. shown below

Incident pressure (psig)	Mounting position	Protected chamber (cu ft)	Closure time (msec)	Maximum pressure in chamber (psig)	Total leakage (cu ft)	Average leakage rate (cu ft/sec)
63	vertical	275	31	4.6	97	3130
8	vertical	275	35	4.4	93	2650
7.9	horizontal[a]	13.2	35	8.1	8.2	----
3.9	horizontal[a]	13.2	78	4.6	4.7	----

Remote actuation: started in 90 msec, closed in 50 msec; total closing time = 140 msec

[a]Subject to reflected pressures.

water gage pressure drop, and may be used for air intake or exhaust service. The valve is closed by the seating of a 16-in. diameter disk traveling through a distance of 4 in.

The valves may also be used, without the remote devices and compression spring release for closing, as blast-actuated poppets. These would be blast-actuated valves with spring-closing for exhaust and spring-opening for intake applications under normal operation.

Several of the valves in battery may be used for large air flows as shown in Fig. 10b. Triggering of the valves to initiate closing is accomplished by explosive capsules fired from a remote nuclear attack detection control station. The impulse of the charge provides sufficient torque to rotate the camshaft, thereby tripping the locking mechanism which maintains the valves in their open positions. This action simultaneously releases the compression of the closure springs, seating the valve disks. Hand operation is through reduction gear for opening and closing. Studies aimed at the further improvement of blast valves are currently in progress.

The intake closure shown in Fig. 11 is being used on a current project and eliminates the need for multiple ports and complicated mechanisms for large air requirements. The 4-ft 6-in. diameter steel plate vent door, 14 in. above the opening, moves through a distance of 12 in. before contacting and compressing the seal material in closing. The door opening is 2 ft above grade and the remainder of the structure is underground. The door is closed by remote actuation of a pneumatic system, with 300 psig air forcing the cylinder down. Closure time is about 1-1/2 sec.

The valves shown in Fig. 6, b and c, may be readily adapted to remote actuation.

Remote-actuated valves are dependent on external sensing devices. The Signal Corps has developed a pair of instruments that are sensitive to thermal and initial gamma radiation[13] of a distant nuclear explosion and that will react electrically to trigger shut the protective closures. This system is furnished by the Signal Corps as "Radiation Detection and Alarm System, AN/FJW-1(V)." A blast pressure detection device designed by Ballistic Research Laboratories and available as part of the system is useful where the direction of blast travel is predetermined. For example, the blast-sensitive unit might be set at the portal of a tunnel or at the top of a shaft to actuate a valve a given distance on the interior, as shown in Fig. 12.

The use of remote-actuated valves eliminates certain problems and creates others. One of the new considerations is arrival time. It can be seen (Fig. 13a) that a closure time of about 200 msec is acceptable for a hardness of 300 psi for a 1-megaton burst but not for smaller weapons. Thus, an over-all design criteria based on a 20-megaton or greater weapon may be conservative for structural design, shock mounting, etc., and still be inadequate for closure design, the arrival time for a given pressure level for smaller weapons being much shorter than for larger weapons.

When the arrival time gets too short for efficient blast valve design of a given size, the path of the blast may be extended, particularly in deep underground installations, by placing the blast valves at the bottom or interior end of the ventilation shaft, as shown in Fig. 12.

Arrival time may also be too long under some circumstances. For example, for an activator sensitive to thermal or gamma radiation at the 2 psig incident pressure from a 20-megaton weapon, the valve would be closed

50 sec before arrival of the pressure and remain closed for at least 12 sec during the positive phase, for a total button-up time of at least 62 sec. The button-up time is increased by about 45 sec if the negative phase duration is added (see Fig. 13, a, b, and c).

When necessary, the arrival time portion of the button-up period may be greatly reduced or eliminated on both intakes and exhausts. In addition, as described below, exhausts can operate continuously through the negative phase.

For installations where combustion-type equipment, such as diesel-electric generators, must operate continuously, it is sometimes possible to draw combustion air from and also exhaust into the interior of the hardened structure. However, except in extremely large structures, it is desirable to minimize this type of operation. Blast-actuated valves can be used on the combustion air intakes to eliminate the arrival time from the button-up period.

Similar devices may be used to permit the exhaust gases to exit prior to the actual arrival of the blast wave. During the positive phase the duration of pressures greater than 2 psi will not exceed approximately 15 sec for weapons up to 20 megatons (Fig. 13b). During this period exhaust gases must be exhausted to the interior or to a protected plenum. During the negative phase (Fig. 13c),[4] exhaust gases may be exhausted to the surface through an automatically throttled opening which will maintain a positive pressure at the engine exhaust, as required to sustain operation (Fig. 14). If the negative phase pressures, which do not exceed 4 psi, are not critical, the throttling device can be eliminated.

Although it is theoretically possible to eliminate even the positive phase "button-up" time on exhausts by exhausting at high pressure through a chamber, as shown in Fig. 15, the power requirement and added equipment costs make this prohibitive in most cases, even though the system may be arranged so that the additional power requirement occurs only under attack or during tests.

Another problem associated with remote-actuated valves which occurs under multiple attack design criteria is the hardening of the exposed electronic detection system, including sensing devices, equipment, and circuits. This deserves serious study in any hard installation design.

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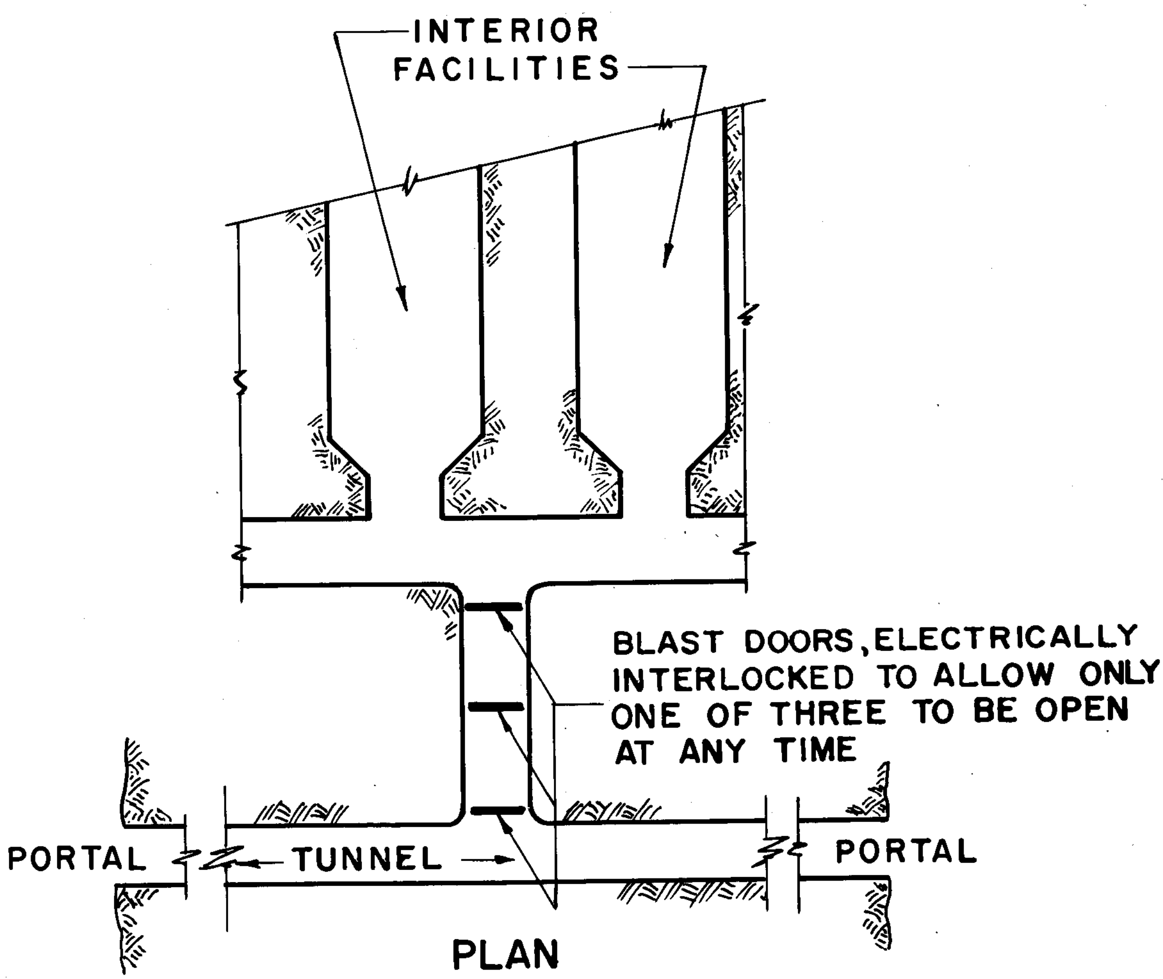

ELECTRICALLY INTERLOCKED ENTRANCE

Fig. 1

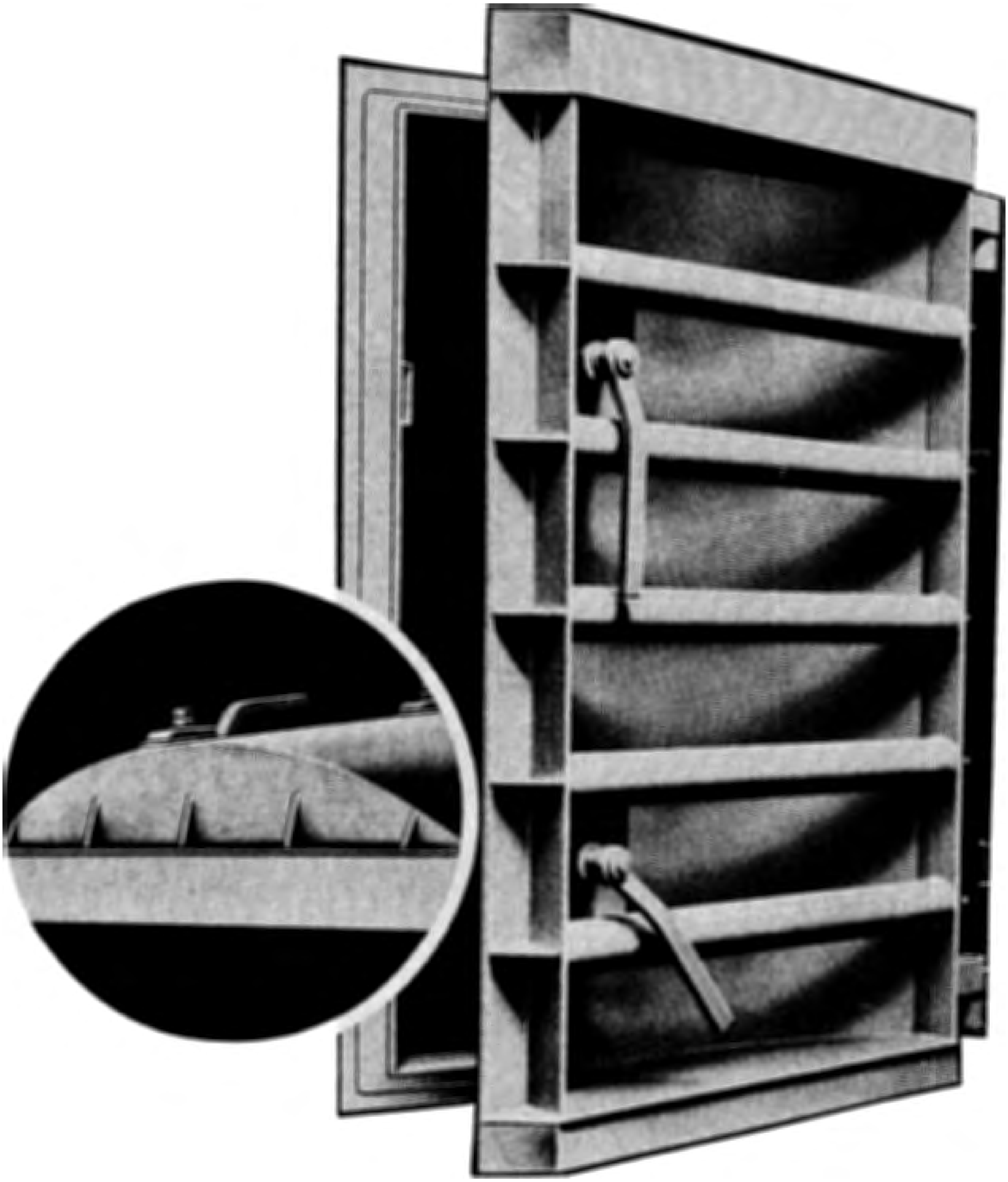

Fig. 2a — Tension arch type blast door

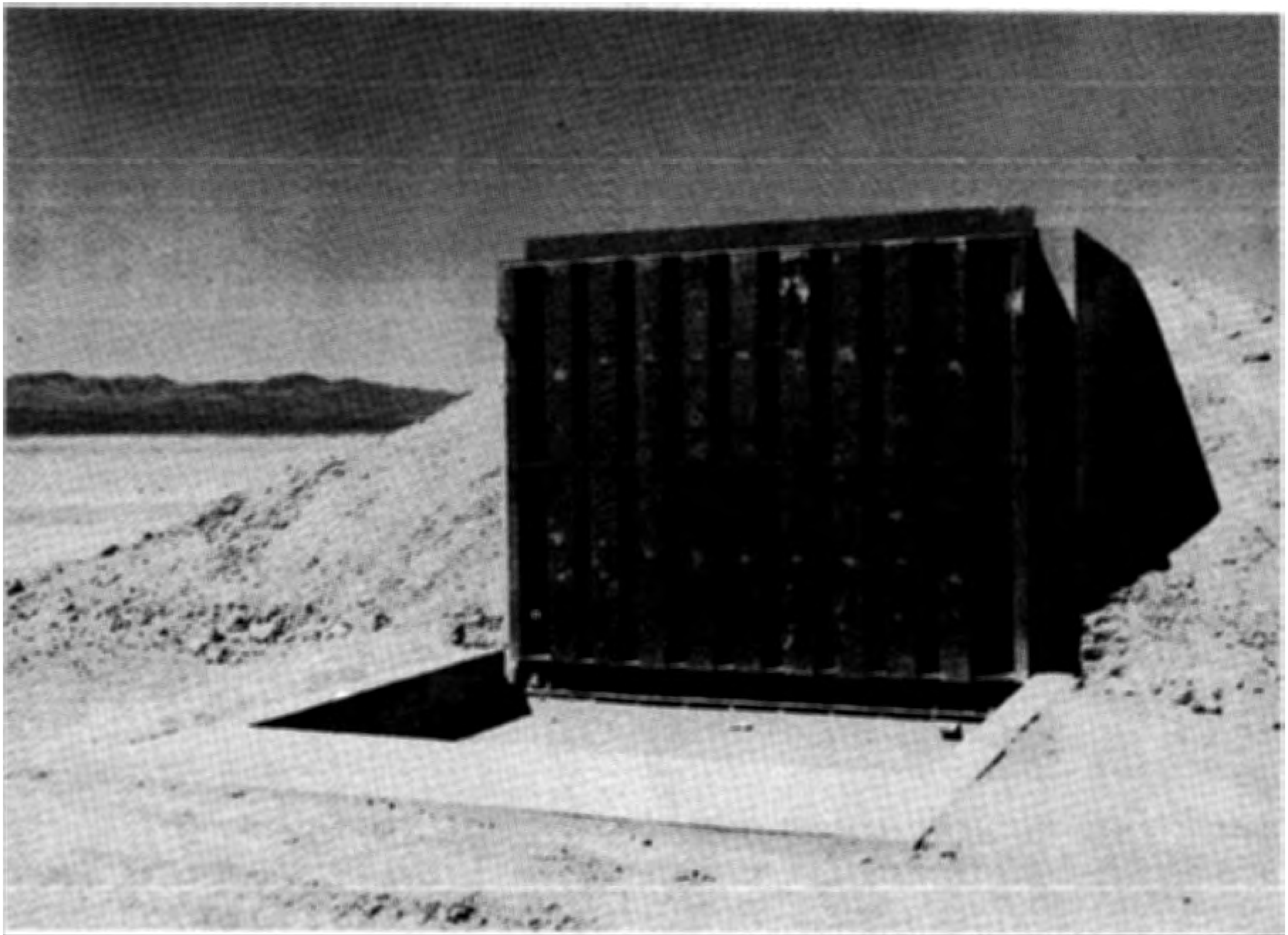

Fig. 2b — Pre-test photograph of draw bridge type door for mass shelter

Fig. 2c—Vault-type door

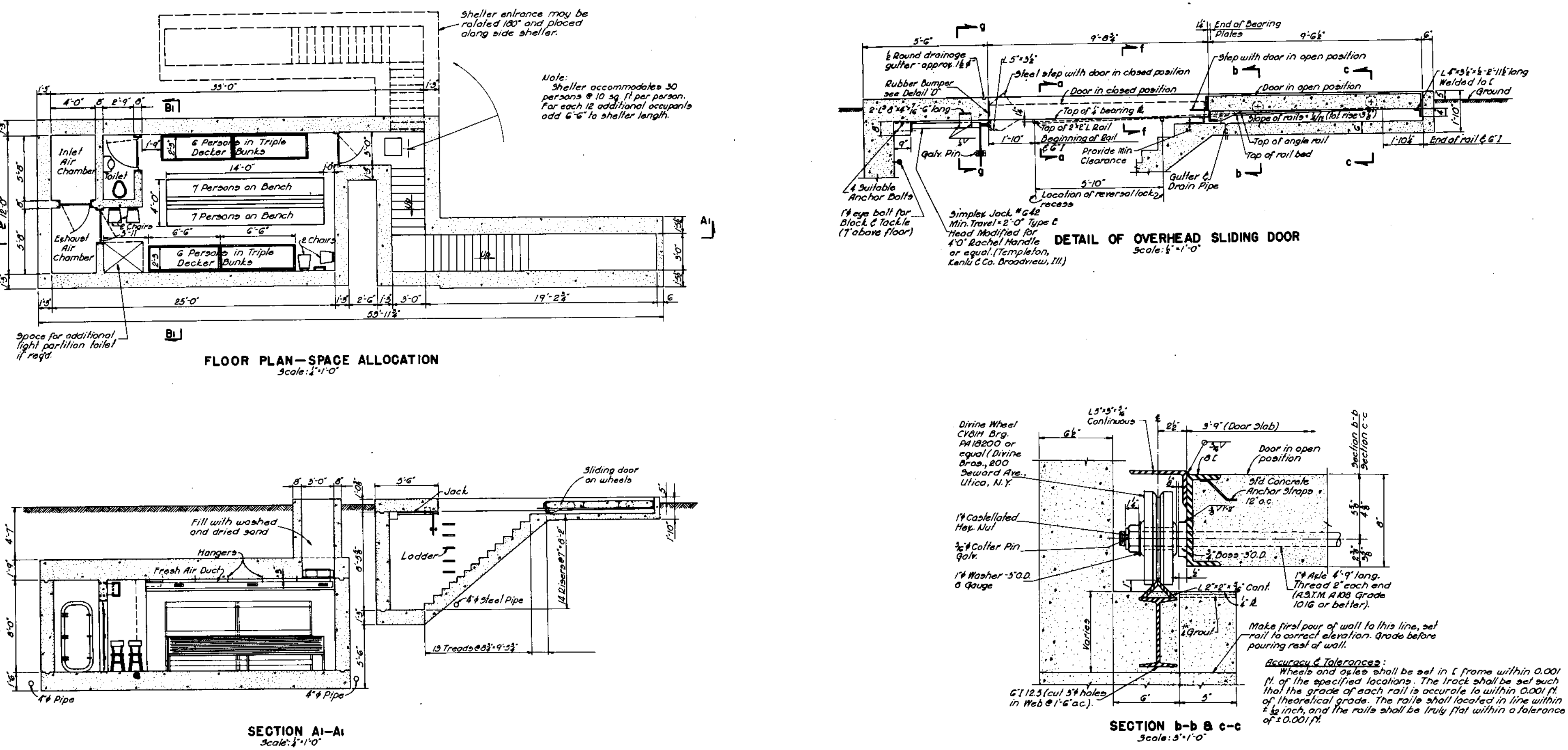

Fig. 2d — Combination reinforced concrete and structural steel door

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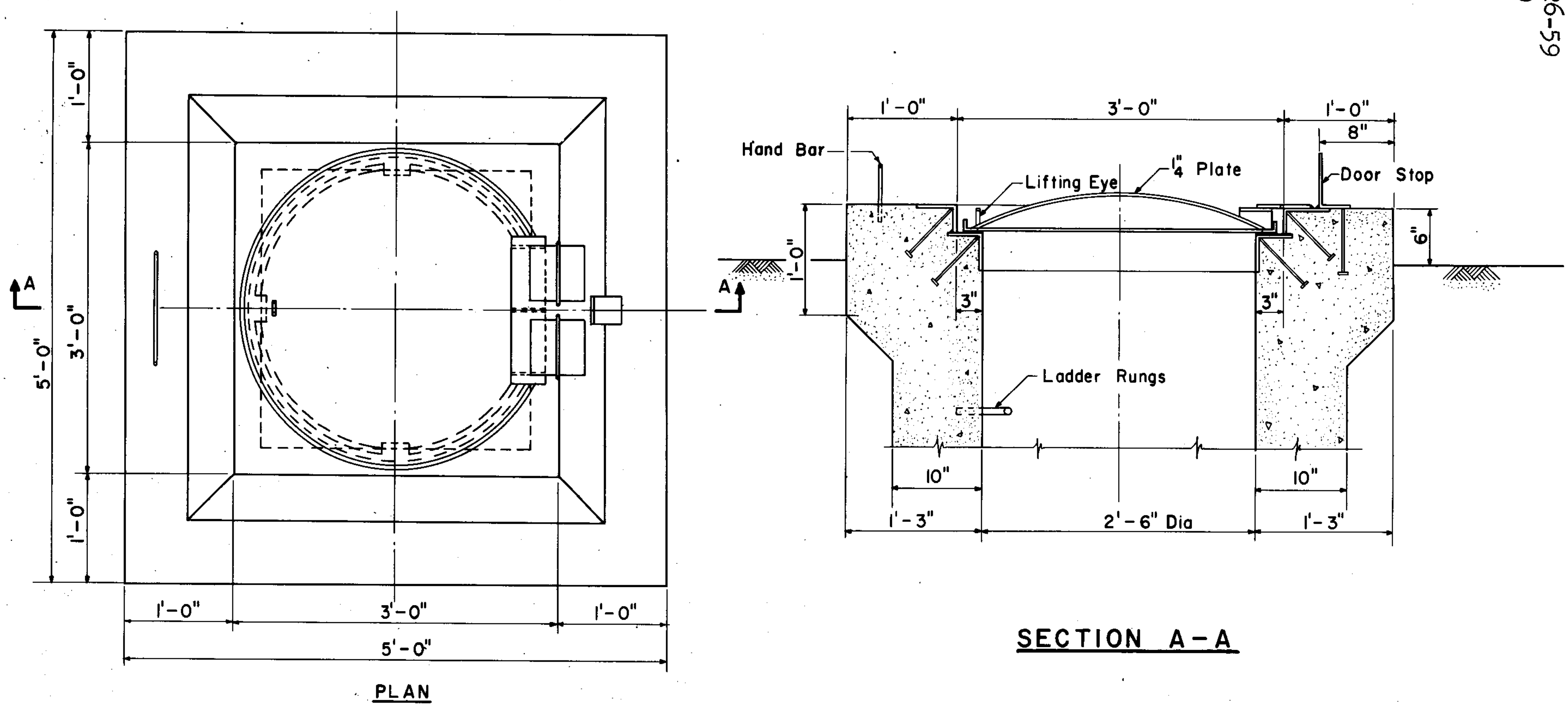
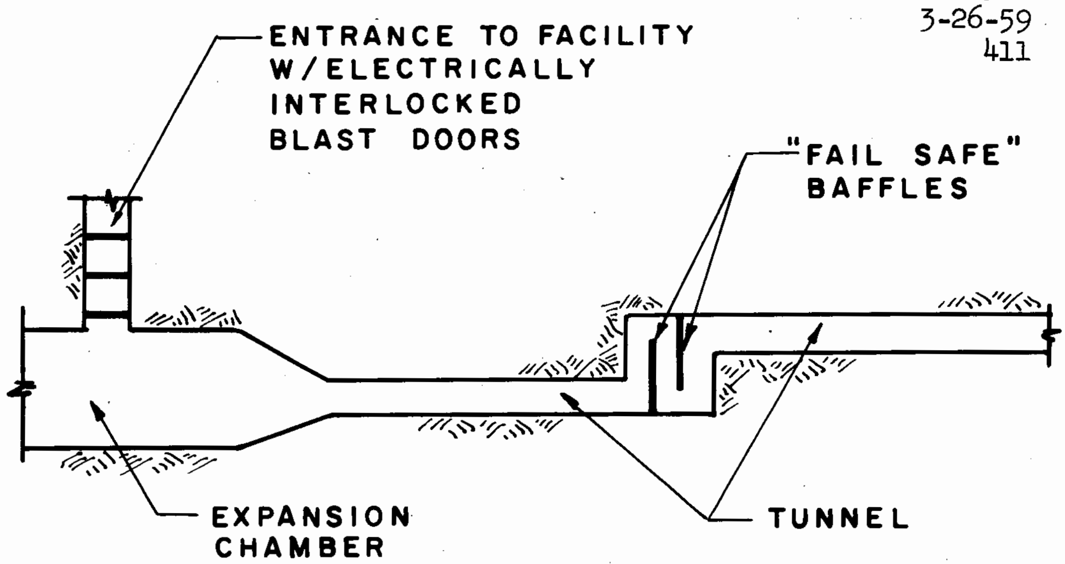
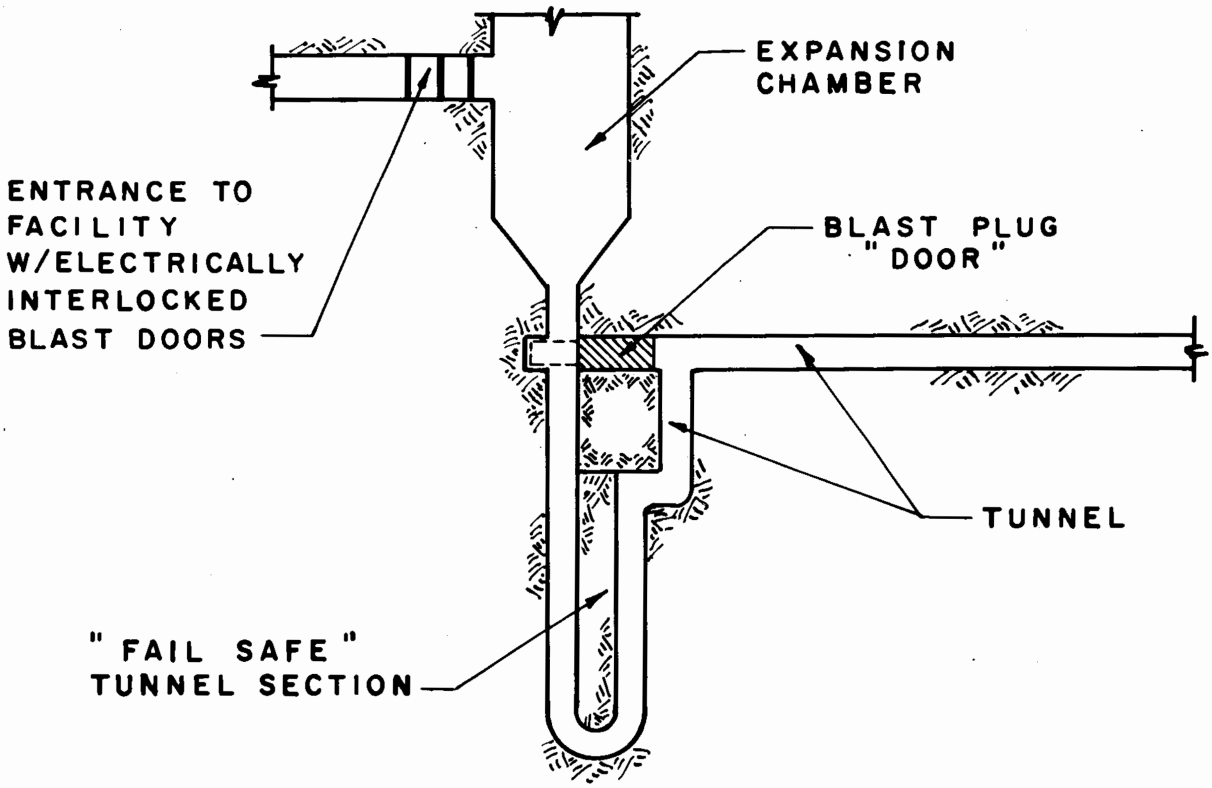


Fig. 3 — Dome plate blast cover for emergency exit



PLAN

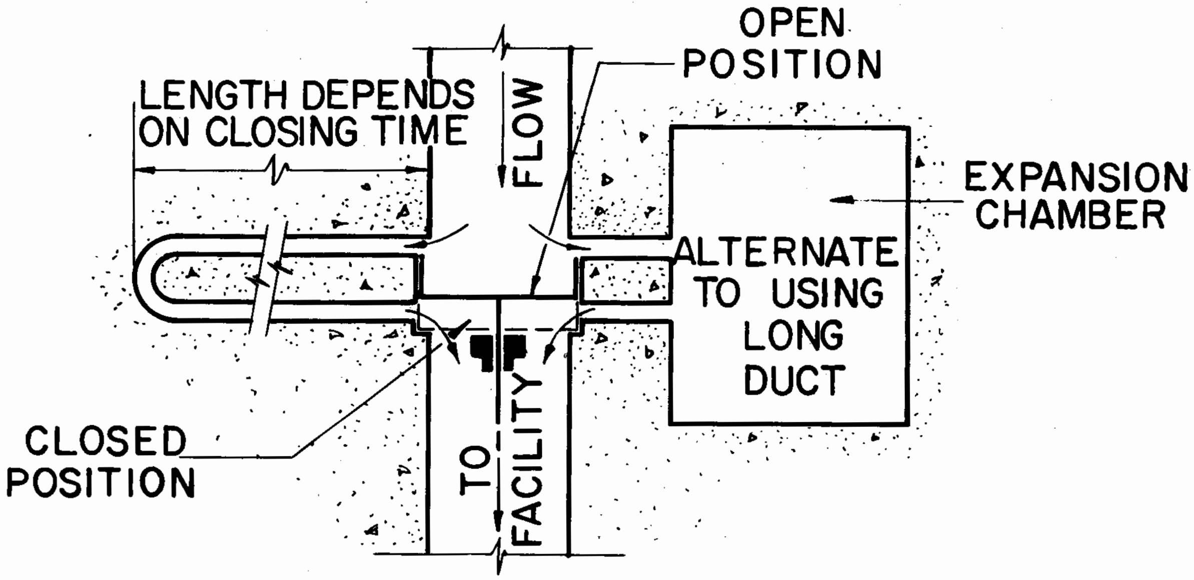
(b) "FAIL SAFE" BAFFLE



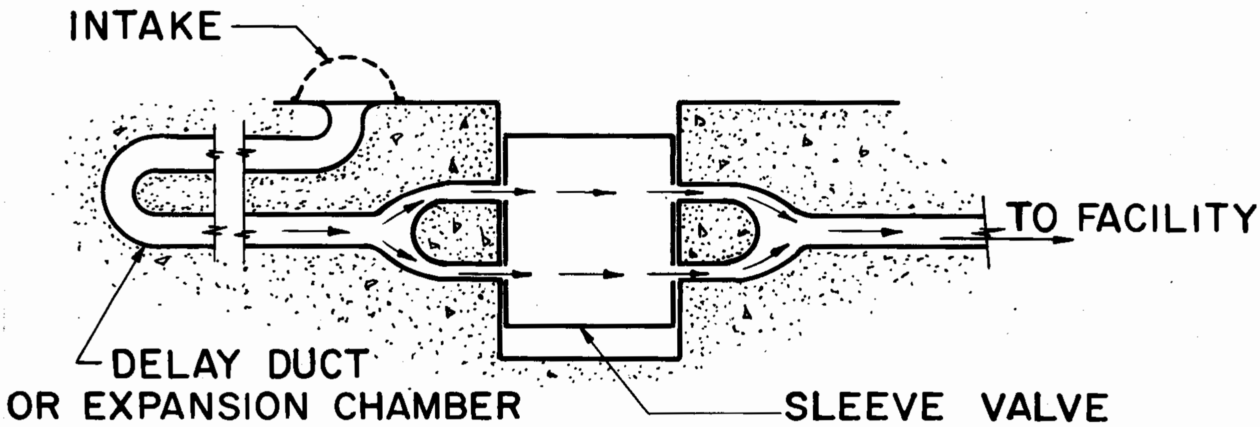
PLAN

(a) "FAIL SAFE" TUNNEL SECTION

Fig. 4

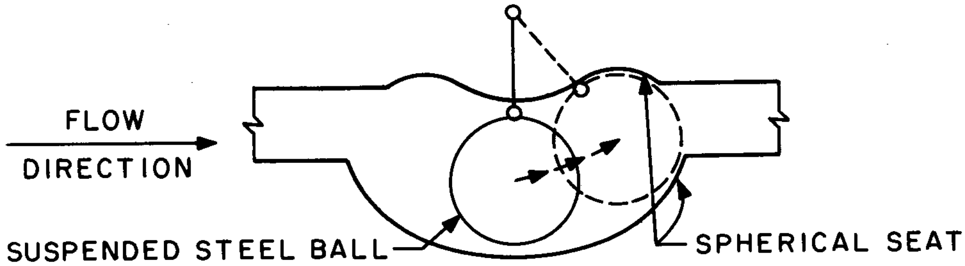


(a) POSITIVE SEALING BLAST ACTUATED VALVE

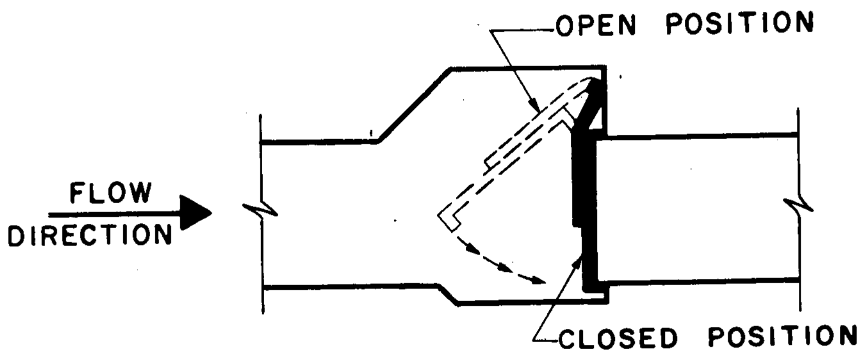


(b) MODIFIED SLEEVE TYPE VALVE

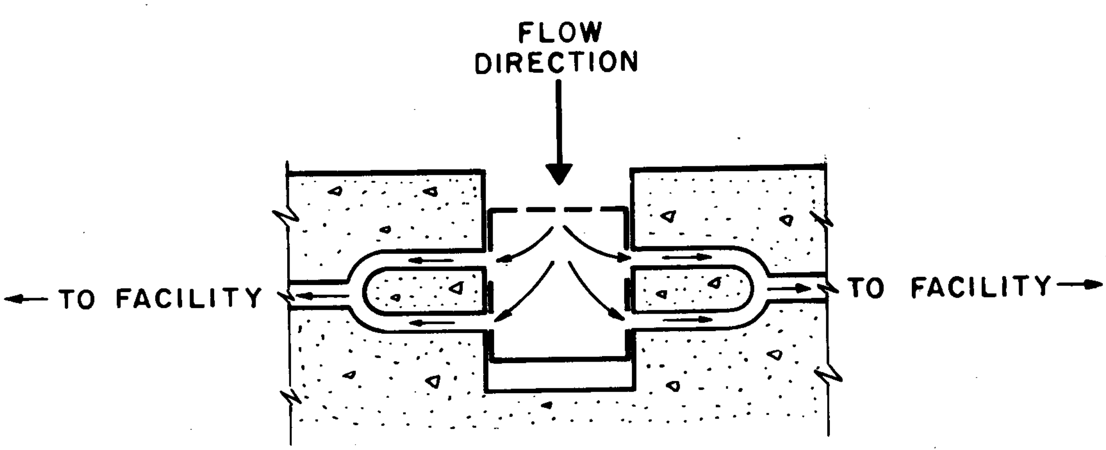
Fig. 5



(a) BALL CHECK VALVE



(b) SWING CHECK VALVE



(c) SLEEVE TYPE VALVE

Fig. 6

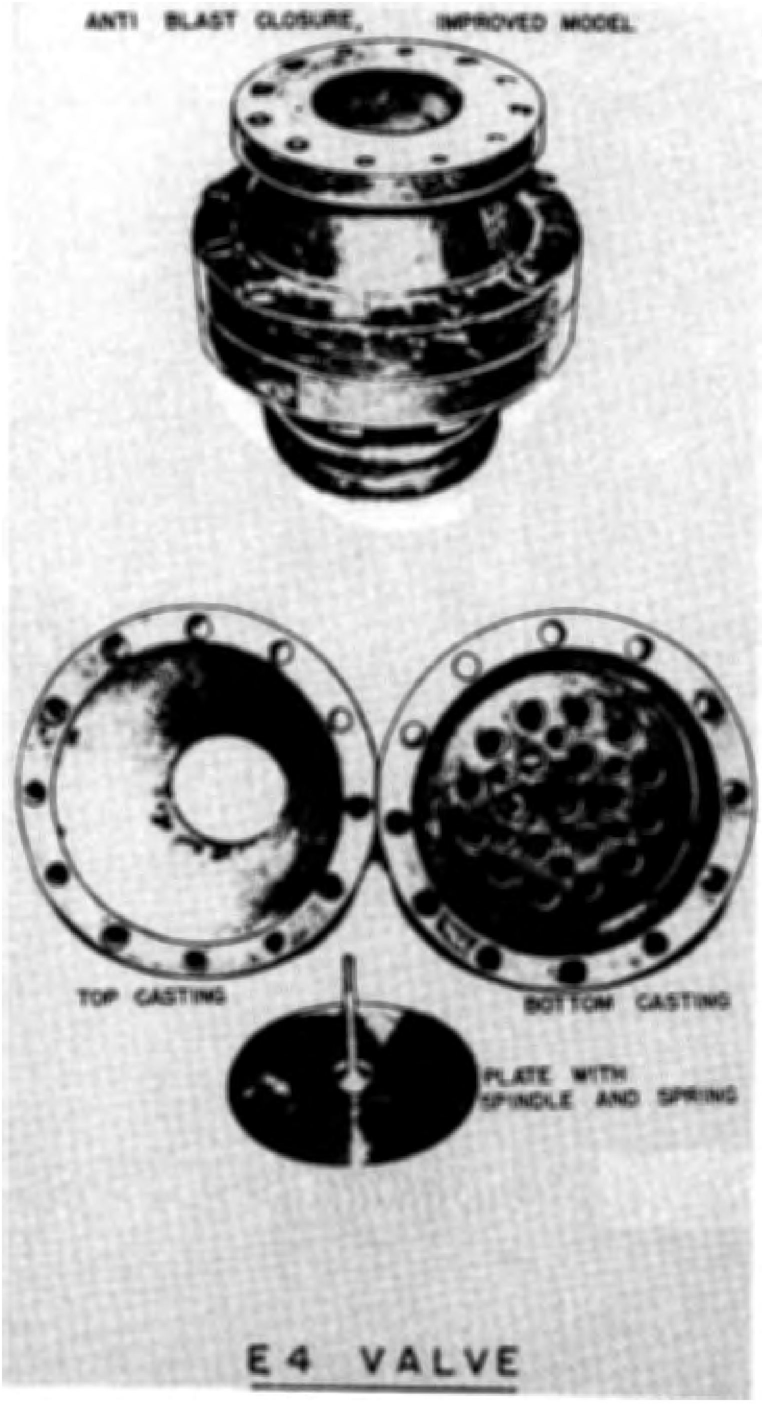
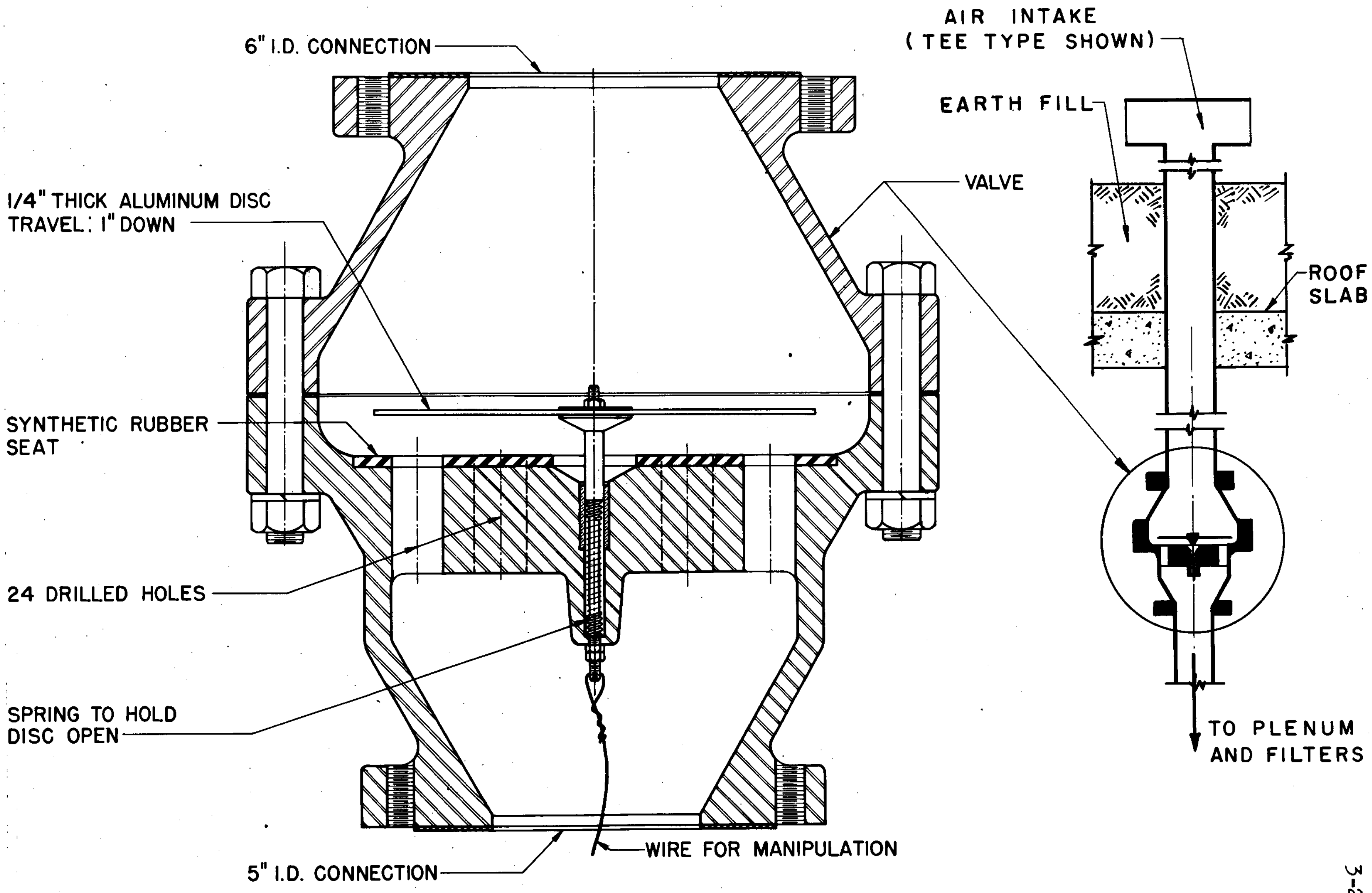

ANTI BLAST CLOSURE, IMPROVED MODEL
TOP CASTING
BOTTOM CASTING
PLATE WITH
SPINDLE AND SPRING
E 4 VALVE

Fig. 7a



E4 VALVE

Fig. 7b

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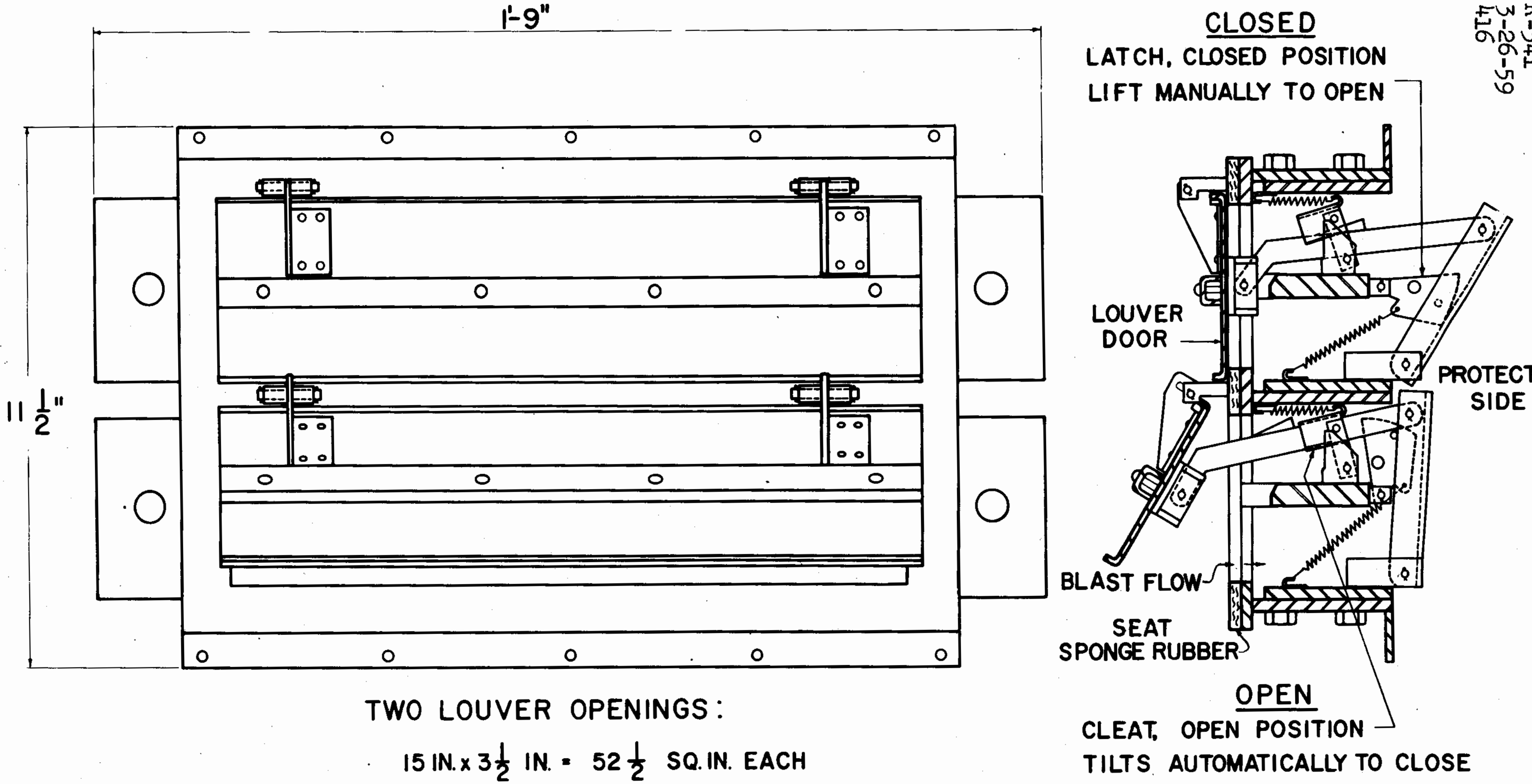
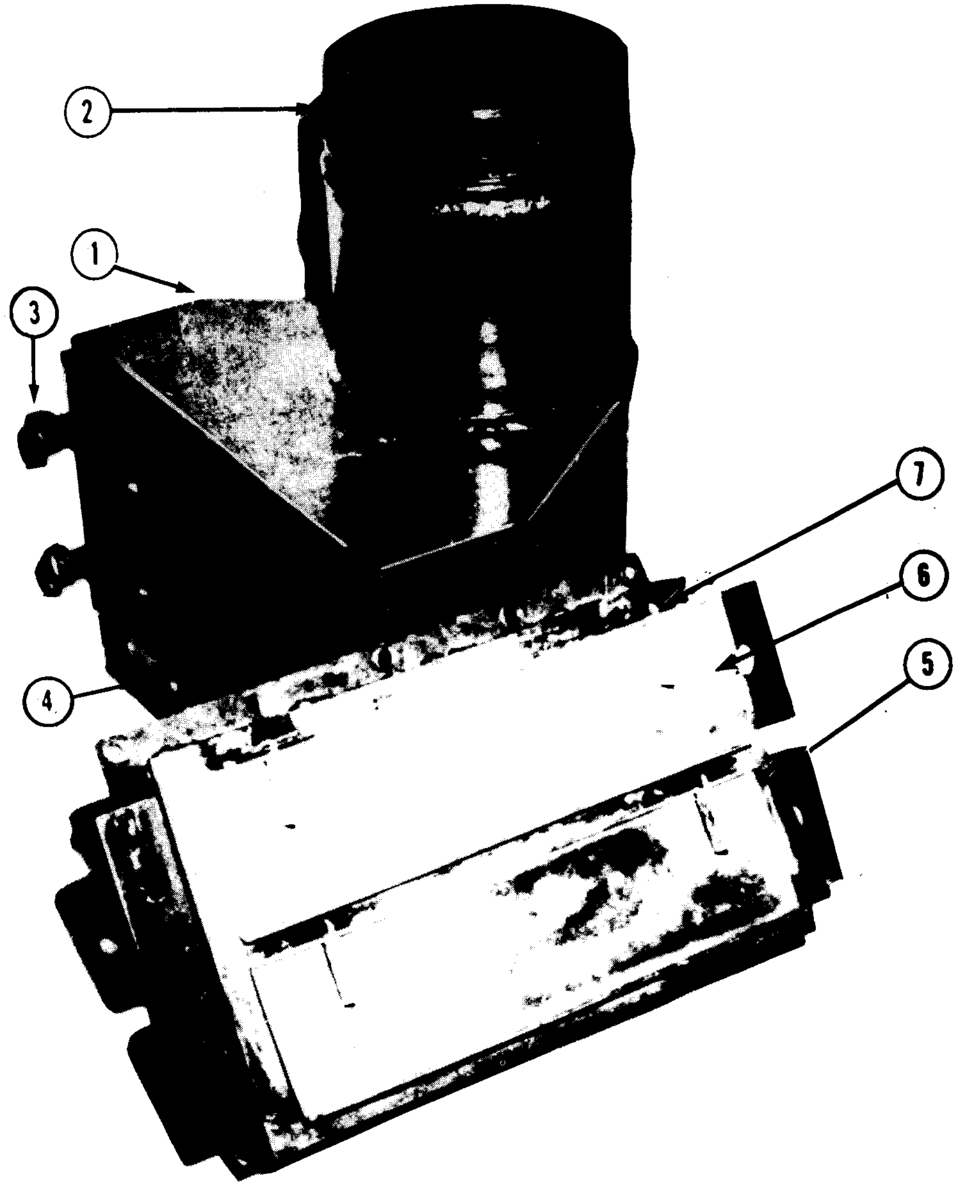


TWO LOUVER OPENINGS:

15 IN. x $3\frac{1}{2}$ IN. = $52\frac{1}{2}$ SQ. IN. EACH

E 19 R1 VALVE ASSEMBLY

Fig. 8a

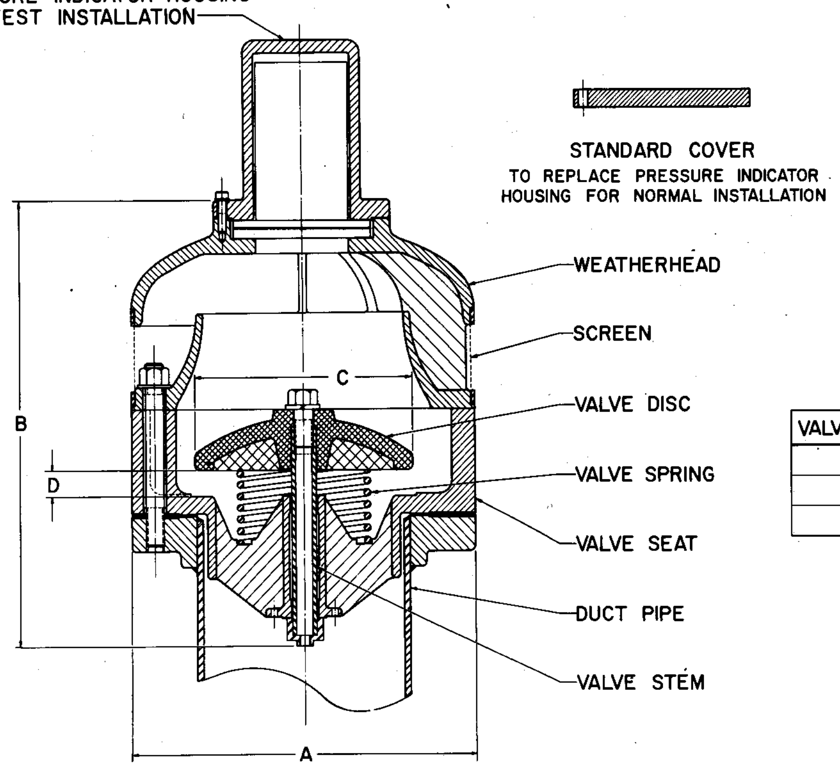


(1) HOUSING
(2) INLET PIPE NIPPLE
(3) CLOSURE ASSEMBLY MOUNTING BOLTS
(4) CLOSURE ASSEMBLY MOUNTING GASKET
(5) CLOSURE ASSEMBLY
(6) LOUVER DOOR
(7) LOUVER DOOR HINGES

RATED CAPACITY - 600 cu ft / min
RESISTANCE AT RATED CAPACITY - 0.4" WG
OVERALL WEIGHT - 130 LB
INLET PIPE CONNECTION - 8" STD PIPE
PLATE TRAVEL TO CLOSE - 30°, 4" RADIUS
DESIGNED FOR 50 PSI
APPROX WEIGHT OF EACH LOUVER DOOR - 0.8 LB

E 19 VALVE, INLET AND DATA

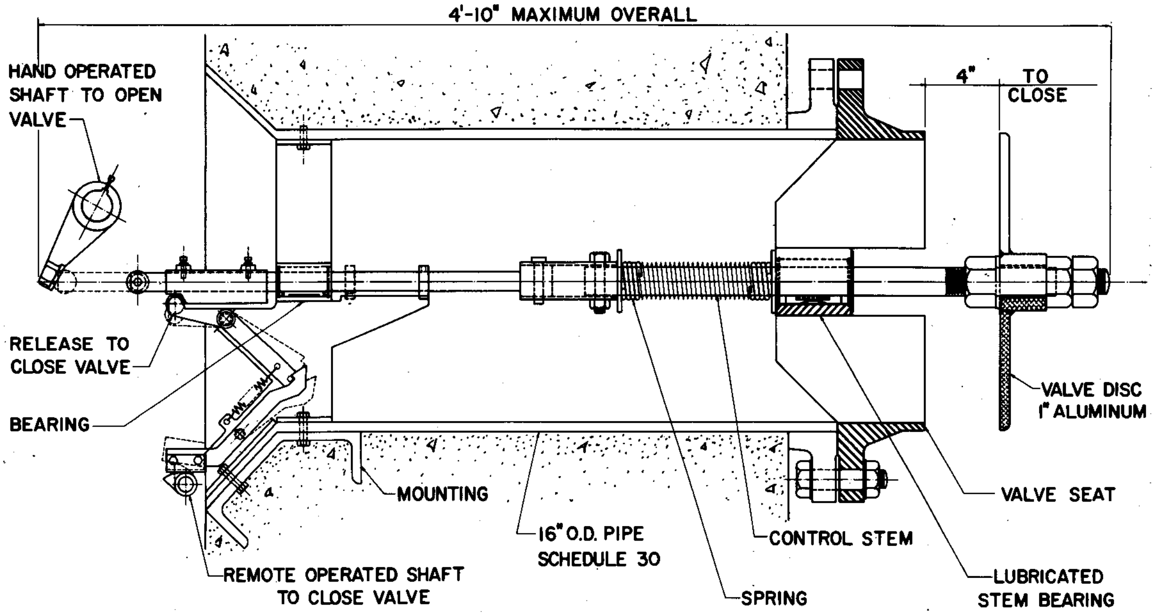
Fig. 8b



VALVE SIZE	A	B	C	D
12"	20½	25¾	13	1.50
16"	23½	29¾	16	2.38
24"	32	42¾	22	3.63

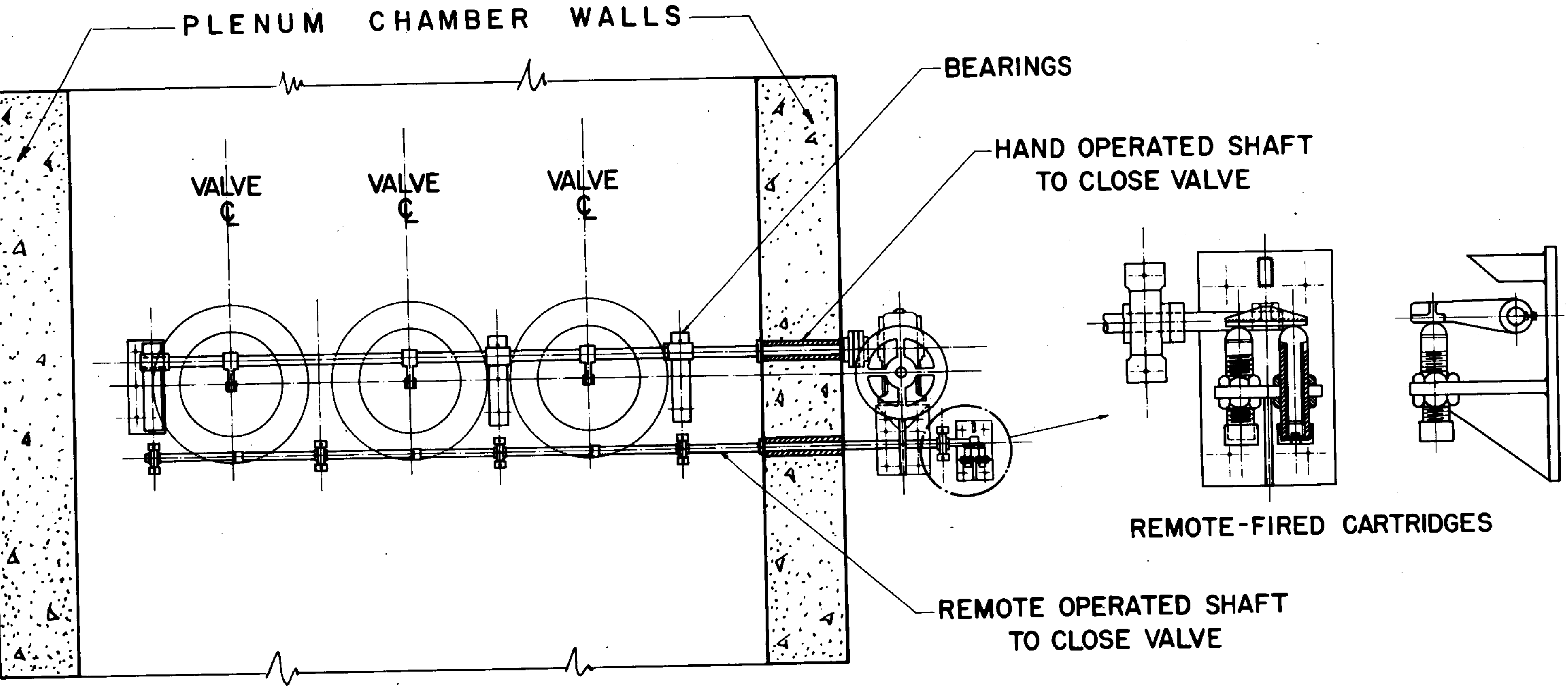
ODCM VALVE ASSEMBLY

Fig. 9



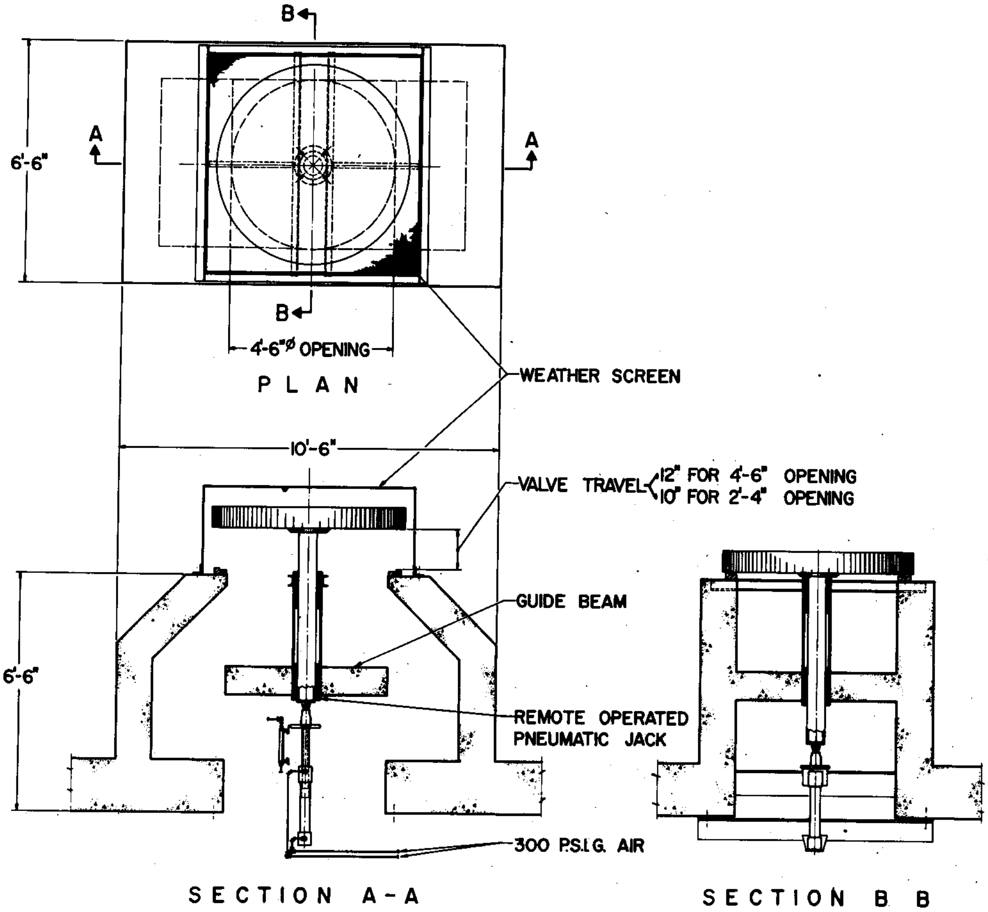
OCE VALVE ASSEMBLY

Fig. 10a



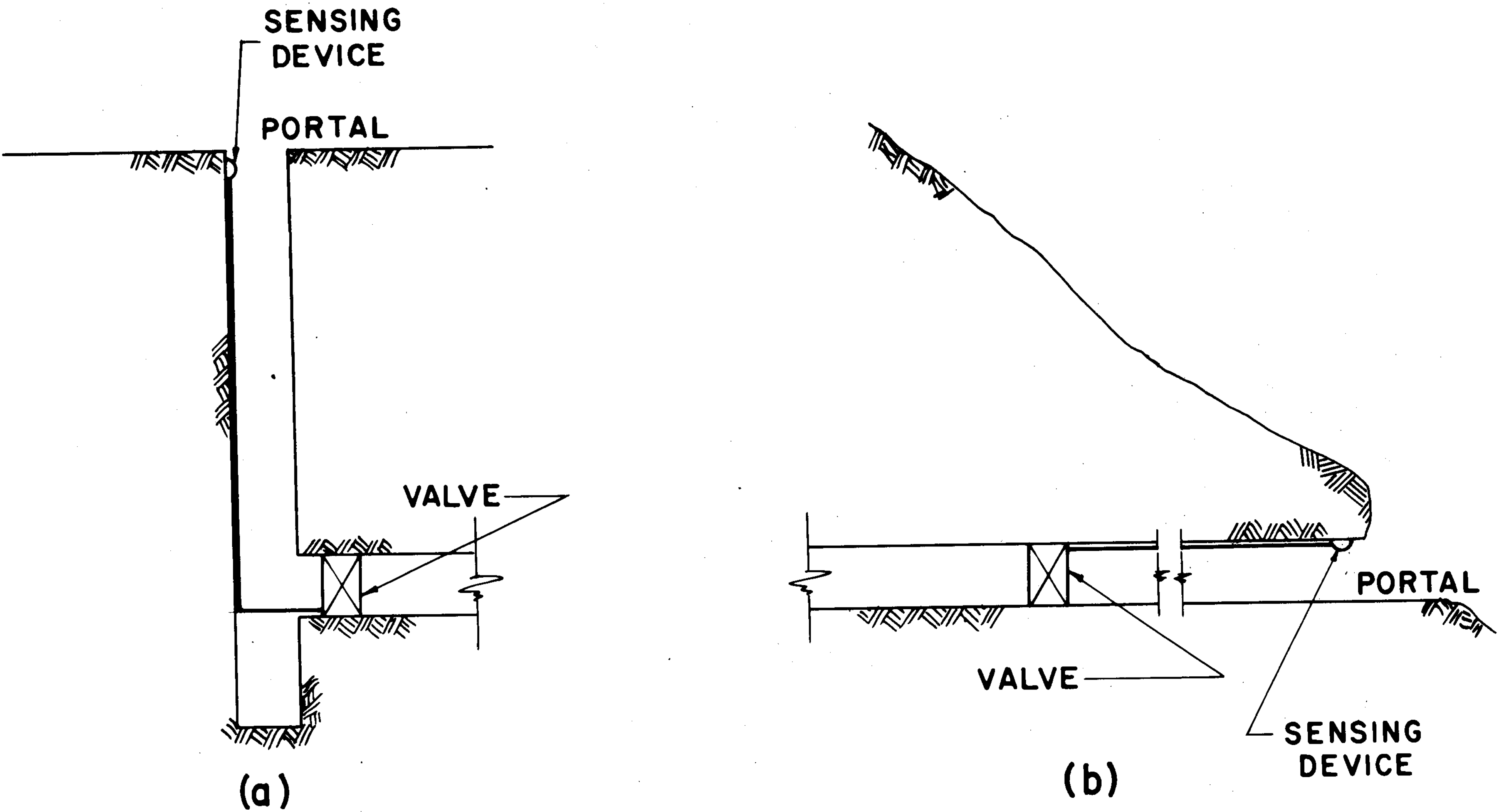
OCE VALVE INSTALLATION

Fig. 10 b



INTAKE CLOSURE

Fig. 11



LOCATION OF DETECTION SYSTEMS

Fig. 12

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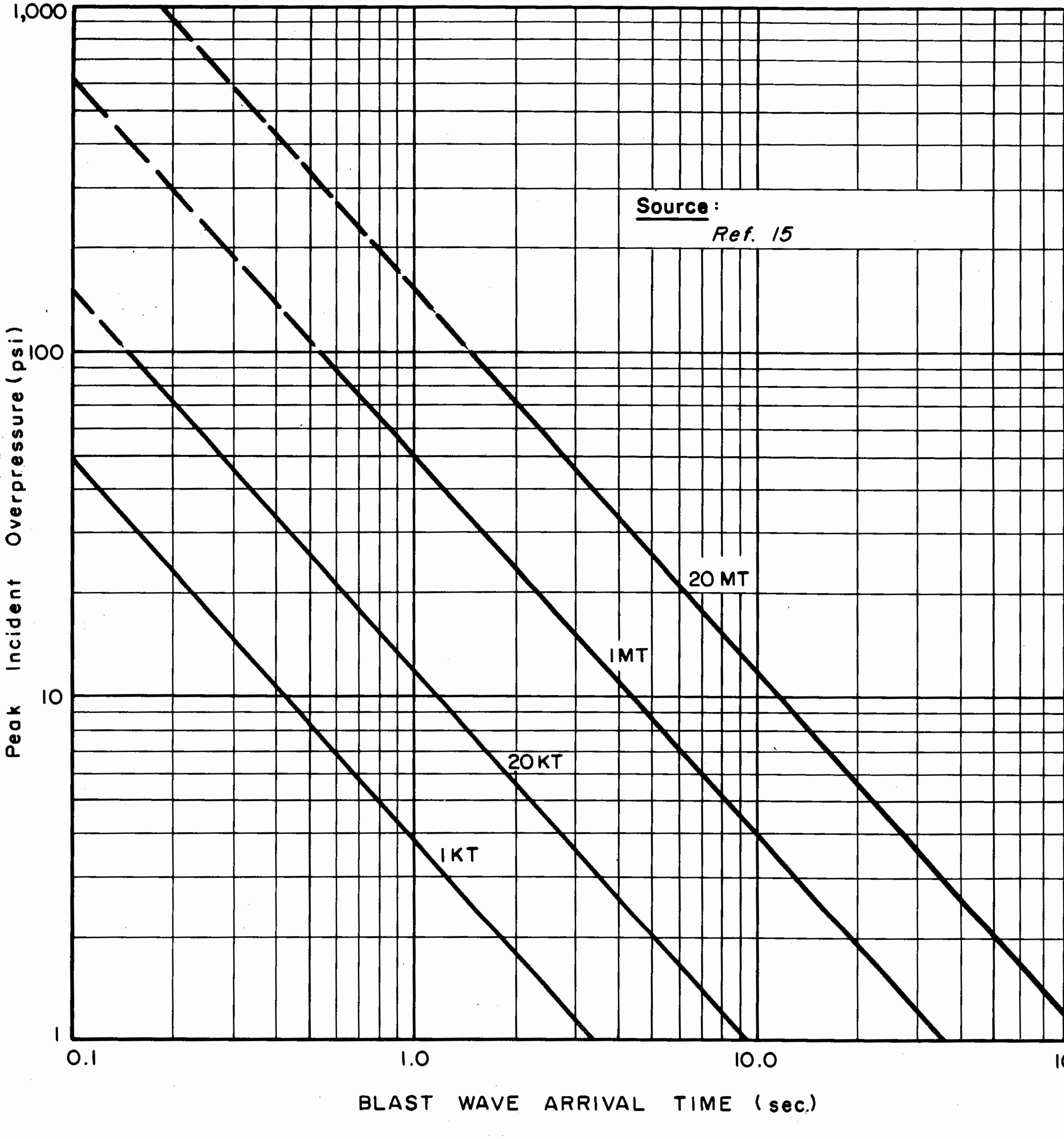

Fig. 13a BLAST WAVE ARRIVAL TIME
FOR VARIOUS WEAPON SIZES
(SURFACE BURST)

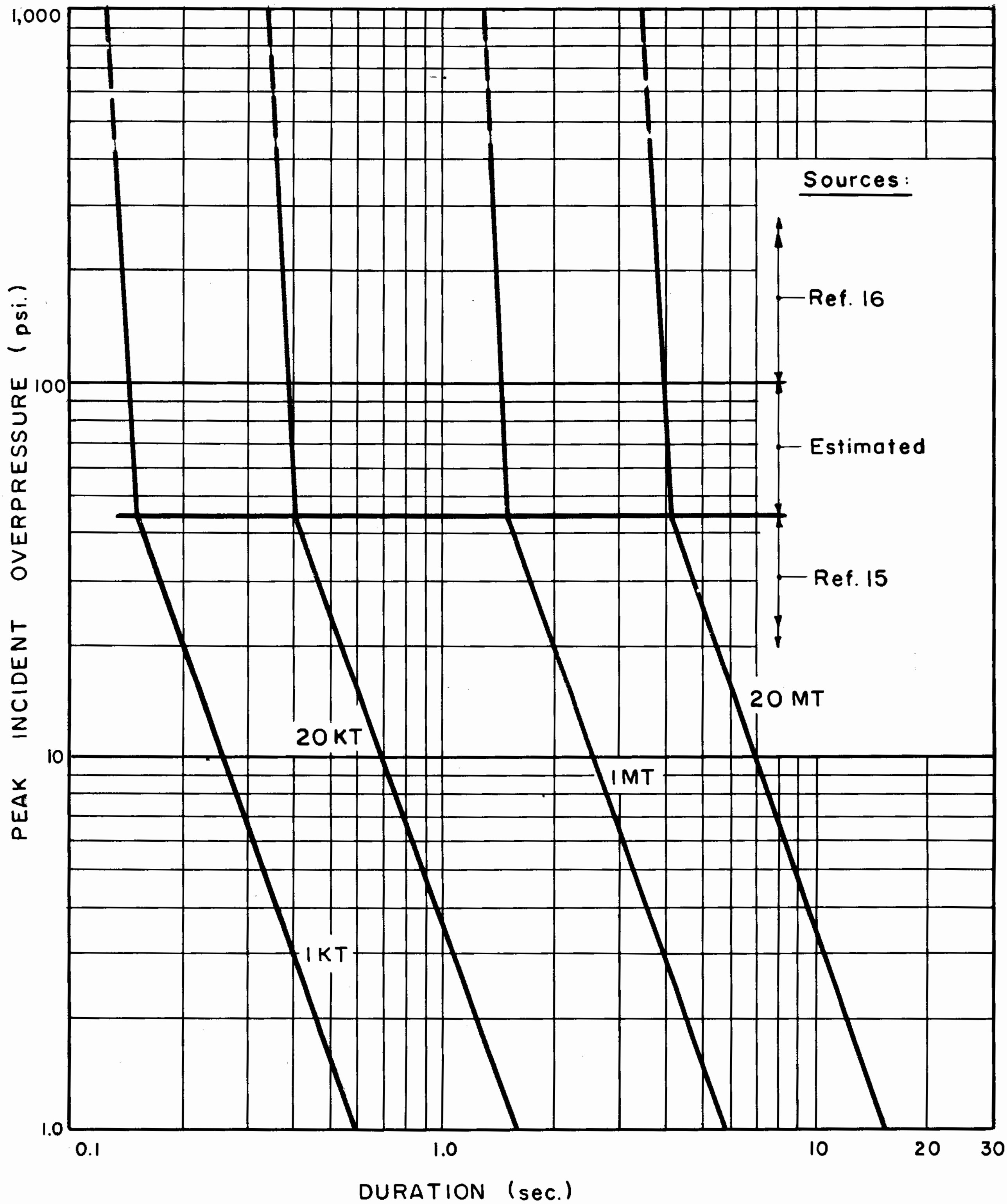

Fig. 13b DURATION OF POSITIVE PHASE FOR VARIOUS WEAPON SIZE (SURFACE BURST)

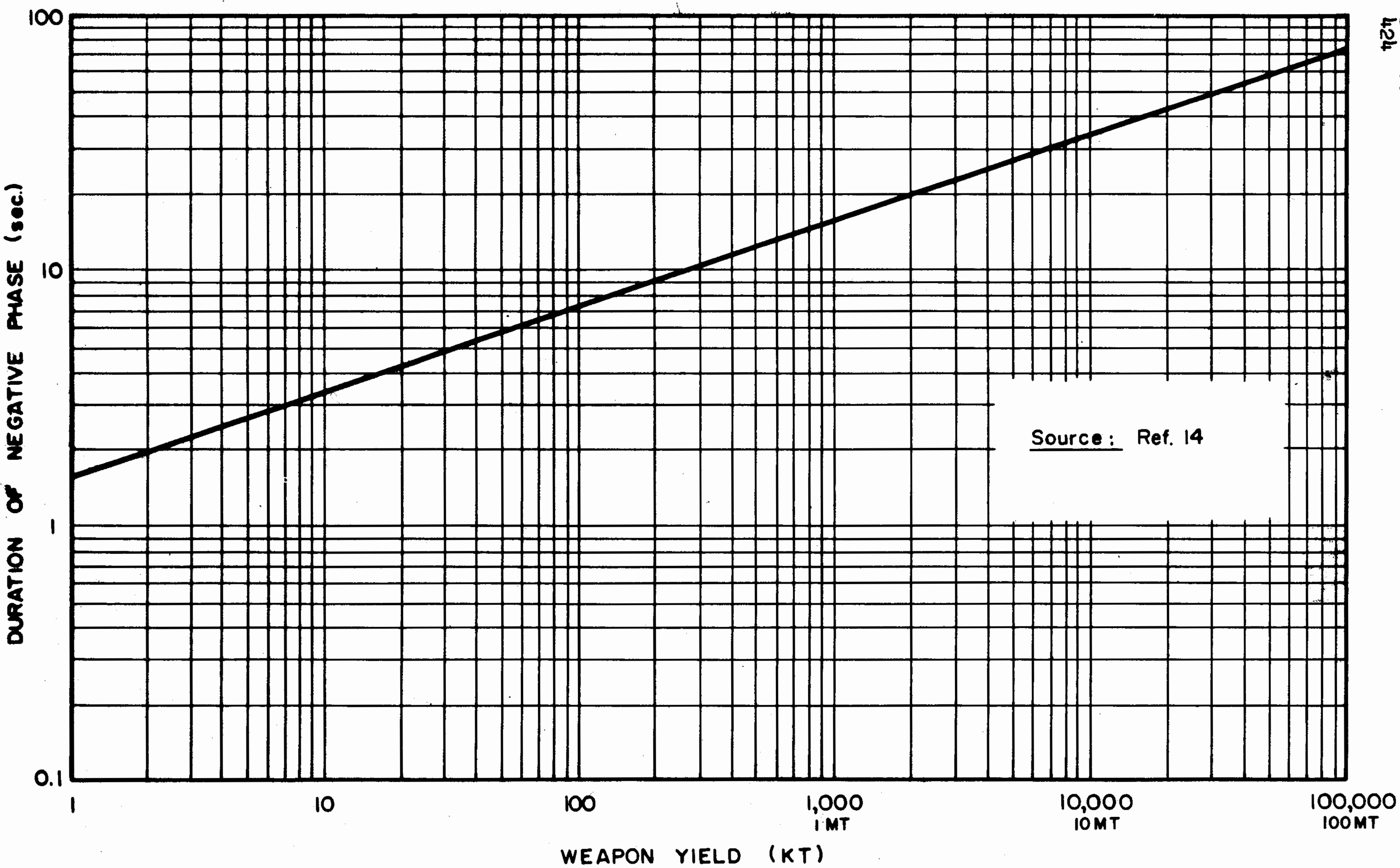

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Fig. 13c DURATION OF NEGATIVE PHASE FOR SURFACE BURST

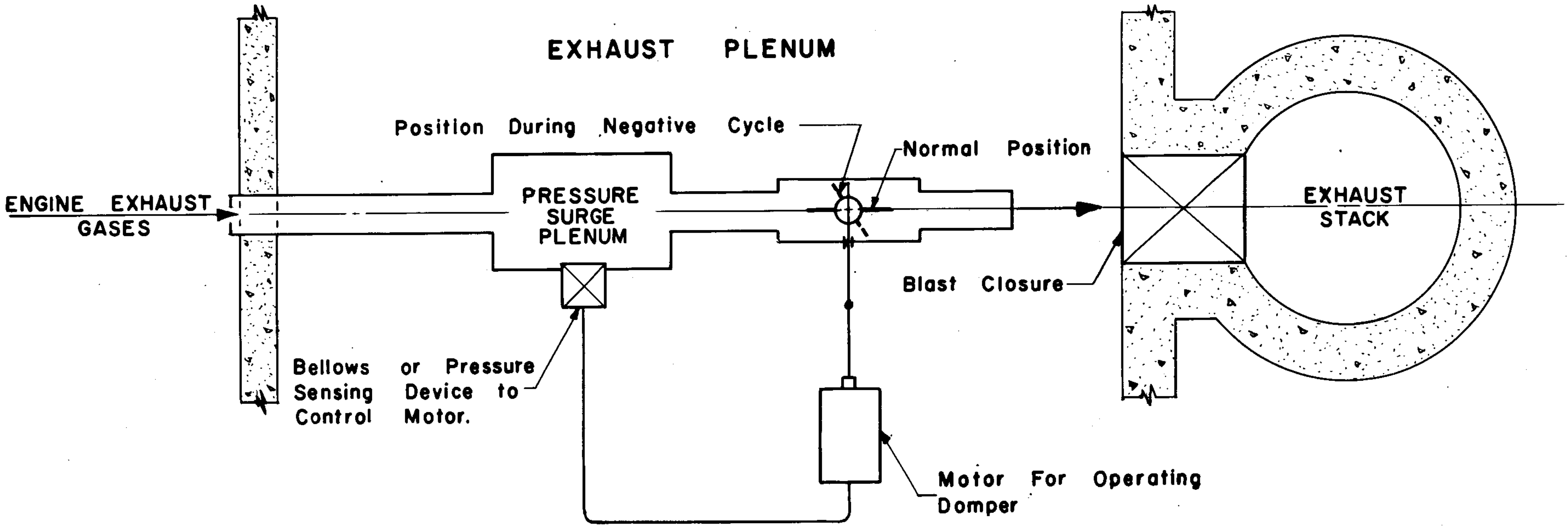

Fig. 14 — Schematic arrangement of exhaust throttle

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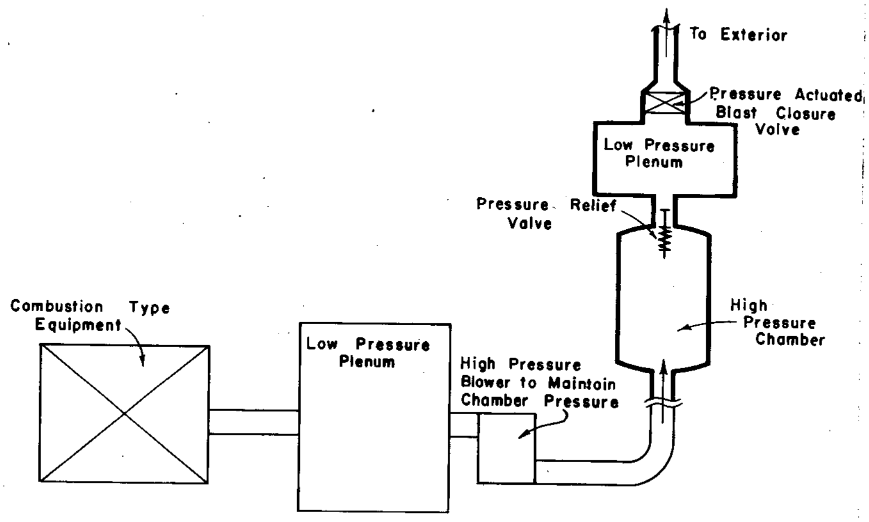

HIGH PRESSURE EXHAUST SYSTEM

Fig. 15

SHOCK WAVE ATTENUATION IN TUNNELS

H. L. Brode
The RAND Corporation

Since any underground installation must have openings to the outside world for normal and probably frequent daily access, their influence on the vulnerability of the complex needs some consideration.

The destruction of the surface opening itself need not cause disruption of the underground operation and need not trap the occupants if multiple entrances are provided. Such multiplicity would depend on both the geology and location of the site and the nature of the mission. In general a sufficient number of openings should exist and should be so located that a single multimegaton explosion cannot destroy more than one of them, and so that the probability of blocking all of them is remote in view of the expected or projected threat. A large number of openings should not be important to such a deep underground complex unless its military mission requires ready access to the surface at all times--since otherwise the openings would not represent attractive targets and would not be subject to specific attack.

An opening which winds up in the crater of a large-yield surface explosion could not be counted on for useful exit or entry for a long period of time--since it would be collapsed and rubble-filled and would be highly radioactive.

An opening exposed to 1000 psi might be expected to be still there afterwards. In any case an important question is: when an opening is exposed to pressures in the thousands of psi what kind and what strength of shock will travel down the tunnel and find its way into the interior?

Here one cannot safely take the usual overpressure vs distance curve and scale it by the contemplated yield since such a curve refers to a burst in air which decays rapidly both from the three-dimensional expansion into a rapidly increasing volume of air and from the heating of this air. In the tunnel case we have the analogy of a shock tube--a one-dimensional expansion--for which the rate of decay of the shock overpressure might be expected to be very slow.

Even in the shock tube there is a certain amount of decay, however, as the shock is dissipated by heating the air in the tube and by losses to the walls through a boundary layer formation.

A simplified linearized equation for shock wave attenuation in a tube (used by Bhargava in his thesis investigation of the effect of wall surface roughness) is as follows

$$\delta(\Delta P_s) = C\Delta P_s \left(\frac{u}{U}\right)\left(\frac{L}{D}\right)\left(\frac{K}{D}\right)$$

$\delta(\Delta P_s)$ is the change or decrease in shock overpressure in psi

C is a "constant" of proportionality

ΔP_s is the initial shock overpressure

u is the particle or mass flow velocity behind the shock

U is the shock velocity

L is the length of tube traversed

D is the diameter of the tube

K is the equivalent wall roughness dimension

This is not a very useful equation for quantitative evaluations over large ranges of any of the variables--in fact the constant C changes from ~ 0.02 for a smooth wall, to ~ 4 for galvanized pipe, to ~ 0.4 for roughened pipe in order to fit the data. Nevertheless, the direction and rough magnitudes of effects is made clear:

1. Attenuation is more rapid for stronger shocks (ΔP_s) and (u/U).
2. The attenuation is an increasing function of tunnel length.
3. The attenuation is enhanced to an important extent by roughness of the tunnel walls, i.e. unlined tunnels would be excellent.

Eventually a shock peters out in a tube just from such attenuations. As a shock progresses along the tube a boundary layer grows behind it--this boundary layer will choke off the laminar flow with turbulent flow finally. Even in a clean, smooth hard-walled shock tube the flow has become choked by the boundary layer growth in a tube length of less than 500 diameters. For a fifteen foot smooth-walled tunnel that would correspond to a mile and a half. However, it is not necessary to go that far to get rid of serious shock effects. That rather represents a limiting distance to which one could propagate a shock in an ideal tunnel.

Some experimental evidence of the shock decay in a four inch shock tube is shown in Fig. 2. Here the smooth stainless steel tube shows a reduction in shock pressure of a third in a length of 150 tube diameters, i.e. a pressure loss more or less proportional to the initial shock strength, as the previous approximate formula indicated. These data were furnished by Dr. Peter Rose of AVCO. For lower shock strength, a somewhat similar set of results were produced by J. D. Day of BRL using a two ft tube. These are shown in Fig. 3.

Another attenuating effect of equal importance is the decay of the peak shock overpressures due to the finite duration of the wave. When a spherical blast wave enters a tunnel its front is followed by a flow field of exponentially decaying pressure.

I have carried out some preliminary calculations of this effect, ignoring the wall losses. The particular geometry considered is that of a strong (1000 psi) shock wave passing over a tunnel mouth at right angles to the tunnel axis--rather than running head on into it. This choice was made both from the standpoint of computational simplicity and from the consideration that since such a case would be somewhat more favorable it could nearly always be arranged that a head-on incidence of blast at the opening could not occur. It would be good sense where possible to avoid constructing a portal so that it faces a likely target area.

If wall loss attenuating effects are ignored and the entering shock is of infinite duration then the blast should travel undiminished down the tunnel--being still 1000 psi after thousands of feet of travel. We have seen that the wall loss effects cause important reductions even in a few hundred feet and cause nearly complete destruction of the shock in less than 1-1/2 miles for a 15 ft tunnel.

Here we see that the finite nature of the blast pulse would attenuate the shock equally fast--as the shock decays from a peak of 1000 psi at the portal to 100 psi after a mile. Since in both cases we have used ideal conditions--in any practical case these combined degrading influences can be expected to reduce an entering shock, by a factor of 10 (from 1000 psi to 100 psi) within a couple of thousand feet, only.

Figure 4 shows somewhat graphically the case described, namely a 10

MT burst at a distance such that the tunnel portal is subjected to 1000 psi. The curves across the top illustrate the advance and decay of the air blast pressures given in psi and distance from the point of burst in thousands of feet.

The full thousand psi does not penetrate far into the tunnel, in fact 800 psi is only 130' inside. The reason is that the full strength of the peak is not maintained down the tunnel at right angles to the flow along the surface--only in the pseudo-static flow case would that be possible. By the time flow is well established in the tunnel the outside pressure is down to less than 500 psi. As the shock progresses down the tube further, the rapid decay of pressure behind the front erodes the peak overpressure itself, thus between 1800 ft and 4900 ft the peak over-pressure drops a factor of two from 200 psi to 100 psi.

A similar decay occurs when the portal peak overpressure is 3000 psi. The peak overpressure drops to 1000 psi within 500 ft, but drops less rapidly thereafter. The crossover between the tunnel pressure and the surface pressure versus range occurs at about 1250 ft.

The time dependence of these tunnel pressures is roughly exponential, as is the outside pressure.

To shorten tunnel requirements and insure positive safety from air blast, doors or other shock attenuating or absorbing devices will probably be advisable. A door at the portal is unattractive for at least two reasons: (1) It must withstand the highest expected portal overpressures while a door somewhere back of the opening need withstand considerably less. (2) If the portal itself lies in the crater, the door will be destroyed before fulfilling its mission--and the tunnel will be exposed to high overpressure shocks.

Mounting doors well back in the tunnel protects the door from cratering action thus preserving the interior of the underground complex while at the same time being required to withstand somewhat lower overpressures.

A door which blocks the tunnel, however, will be subject to peak overpressures considerably in excess of the incident tunnel shock. On normal reflection a 1000 psi shock jumps to 8000 psi, a 3000 psi shock would go to 35,000 and 100 psi shock becomes 480 psi. A 10 psi shock only goes to 25 psi, but in all of these reflections in the confines of a tunnel the reflected pressures do not decay rapidly, dying rather like the initial pressure pulse. So a simple door blocking the tunnel must face pressures several times the expected incident pressures.

By various schemes, these bothersome reflections can be minimized or avoided. Figure 5 shows schematically in plan form various simple devices.

Perhaps the first thing that comes to mind in avoiding dead end reflections is to lead off from a through tunnel for the entrance to the complex--or to construct a sort of double entrance, where the two (or more) entrances themselves may be separated enough to avoid both being destroyed by the same bomb explosion. Here the shock races by the side tunnel, exposing it or its doors to the incident pressure only--with no enhanced pressures from reflections.

Various other schemes are feasible along a single tunnel--such as turning corners, or expanding into large chambers, or reverberating around in a system of baffles and chambers not unlike what an acoustics engineer might employ for sound absorption. Side branches might be gone one better by leading them off more or less in the opposite direction from the incoming blast--

allowing the blast to pass on down a long tunnel or into a large expansion and absorption chamber.

Any mechanical closure system has the disadvantages of possible jamming or malfunction after an explosion, of requiring detailed or awkward procedures for normal opening and closing, of requiring some considerable power and movement of large masses.

The massiveness of a positive closure system is essential for absorbing the high impulses from the blast, but leads all too easily to cumbersome moving parts systems, with slow reactions, and considerable power requirements.

One way to avoid some of this difficulty is to fill and empty a hollow door or the space between two bulkhead doors with a liquid--say water. However, one could avoid all large moving parts with a sloping tunnel such as indicated in Fig. 6. Here one needs only two water valves--one to flood the tunnel and the other to drain it. If water were not plentiful in some cities the water could be recirculated as indicated here, but only a small one-lunger pump would be necessary even with several openings and closings of the "water door" per day.

When the blast strikes the water, say with 1000 psi, this pressure is transmitted through the water but is not felt in the air further down the tunnel because of the poor impedance mismatch. As long as the water is thick enough, i.e. has enough mass to not be accelerated to a high velocity by the pressure impulse, the transmitted pressure will be .0003 of the incident pressure. Thus, 1000 psi incident transmits as 0.3 psi. In order not to drive the water down the tunnel at high speed we may need something like 100 ft of it. From a 10 MT burst, and 1000 psi at the tunnel entrance, if we place a "water door" inside the tunnel at a point where the incident pressure will be 150 psi then the total impulse in the reflected pressure on the water surface will be around 200 psi/sec. Only ten ft of water would make quite a splash--with velocities like 1400 ft/sec, but 100 ft of water--more appropriate for the sloping tunnel notion--would result in initial water velocities of a tenth of that (140 ft/sec) and could be easily dissipated by splash barriers or diverted down a side tunnel into a pit, or allowed to fall through a grating in the tunnel floor into a recirculating reservoir.

Any number of other solutions to the closure problem undoubtedly exist and have equally simple and jam-proof features, and the ideas I have indicated here in a most superficial way are meant only as an advertisement of the fact that the door is still wide open. Designs or suggestions as to how to close it will certainly meet with enthusiastic reception in many quarters where there is the need or desire to get down to earth.

DISCUSSION

MR. SHERWOOD SMITH: I would like to ask what the effect is of the size of the opening when you have the blast wave passing along the surface of the ground and you have an opening extending down vertically; that is, a vertical shaft. Another thing, what is the effect of the size of a vertical shaft on the attenuation?

DR. BRODE: I showed a vertical shaft on the slide simply because the geometry of the illustration came out that way. That could as well be the plan view of the side of a mountain and a horizontal tunnel.

There is a real advantage to preventing the blast from entering the tunnel or shaft head-on, as I indicated, since in "turning the corner" the shock temporarily loses its dynamic inertia. This kind of effect should scale with the shaft dimensions, therefore, the same degradation of peak overpressure due to turning the corner should occur at the same number of tunnel diameters inside the shaft.

Similarly the attenuation due to boundary layer or wall losses should be the same at the same L/D, therefore, at the same number of tunnel diameters down the shaft. I say this because even for the shock tube data the Reynolds Number is very large, so that simple scaling of relative dimensions is valid there and is even more valid as the size is increased.

However, the attenuation due to the pulse shape is not a function of the size of the opening for tunnels of relatively uniform diameter, but is only a function of the yield and peak overpressure level at the entrance.

MR. CHARLES SANDOVAL (RAND Corporation): In the case of an oblique shock hitting this tunnel entrance, would there be any built-up pressure?

DR. BRODE: The shock into the tunnel will be greatest for a shock striking the tunnel entrance directly; that is, entering directly down the tunnel. If it has to turn the corner, if the shock is going at right angles, it will be least. If it is somewhere in between, the build-up will be intermediate.

MR. SANDOVAL: On your water door, when the pressure was reflected from the outer water surface, how does this affect the instantaneous (or is it instantaneous?) increase in pressure according to Pascal's Law?

DR. BRODE: It goes up right along with the reflected pressure, because it is not moving. Unless you have too little water, it goes up.

MR. SANDOVAL: This could be damaging to the walls of your tunnel?

DR. BRODE: It could. But if it was that strong, it probably damaged the walls somewhere earlier on the way into the tunnel. Surely, the pressure will go up by this kind of impressive factor: One thousand psi becomes 8,000 on reflection. In a tunnel, it doesn't drop off any more rapidly than the incident shock does. You get 8,000 psi in the tunnel, which is not much different from what it does outside on the surface. The time of the shock-wave attenuation in the tunnel is very similar to that in free air because it is governed by the free air pressures at the entrance.

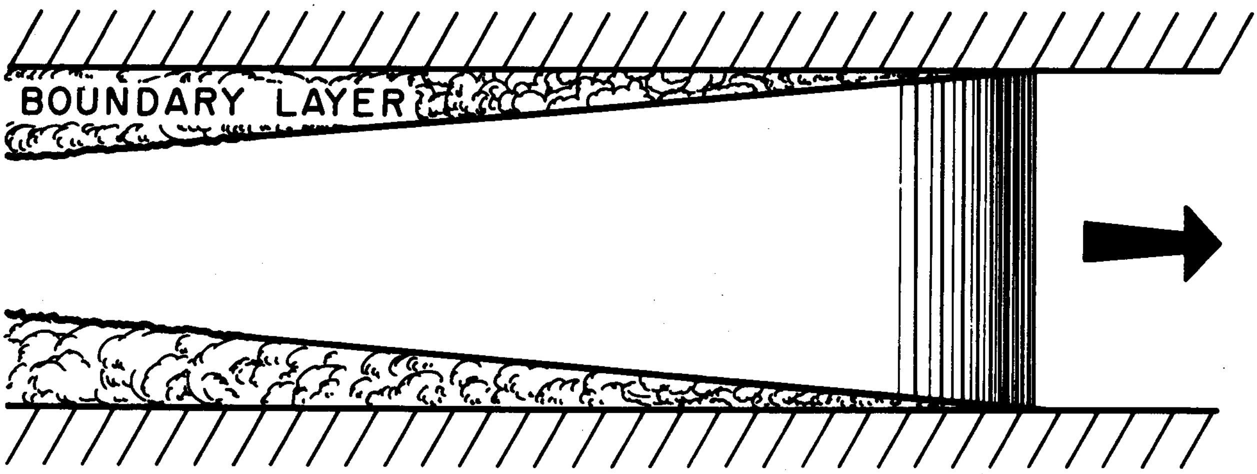

Fig. 1 — Shock tube - boundary layer

$\Delta P \simeq \frac{2}{3} \Delta P_i$ AT L/D = 150

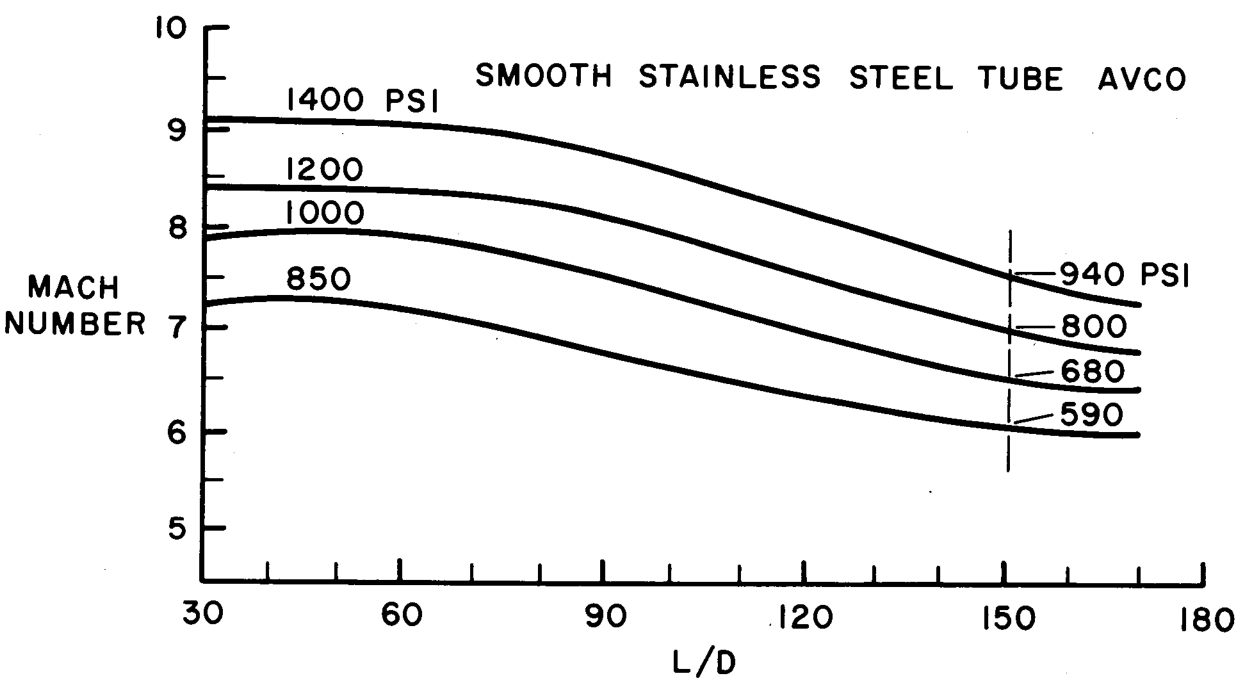

Fig. 2 — Shock tube attenuation data

R-341
3-26-59
438

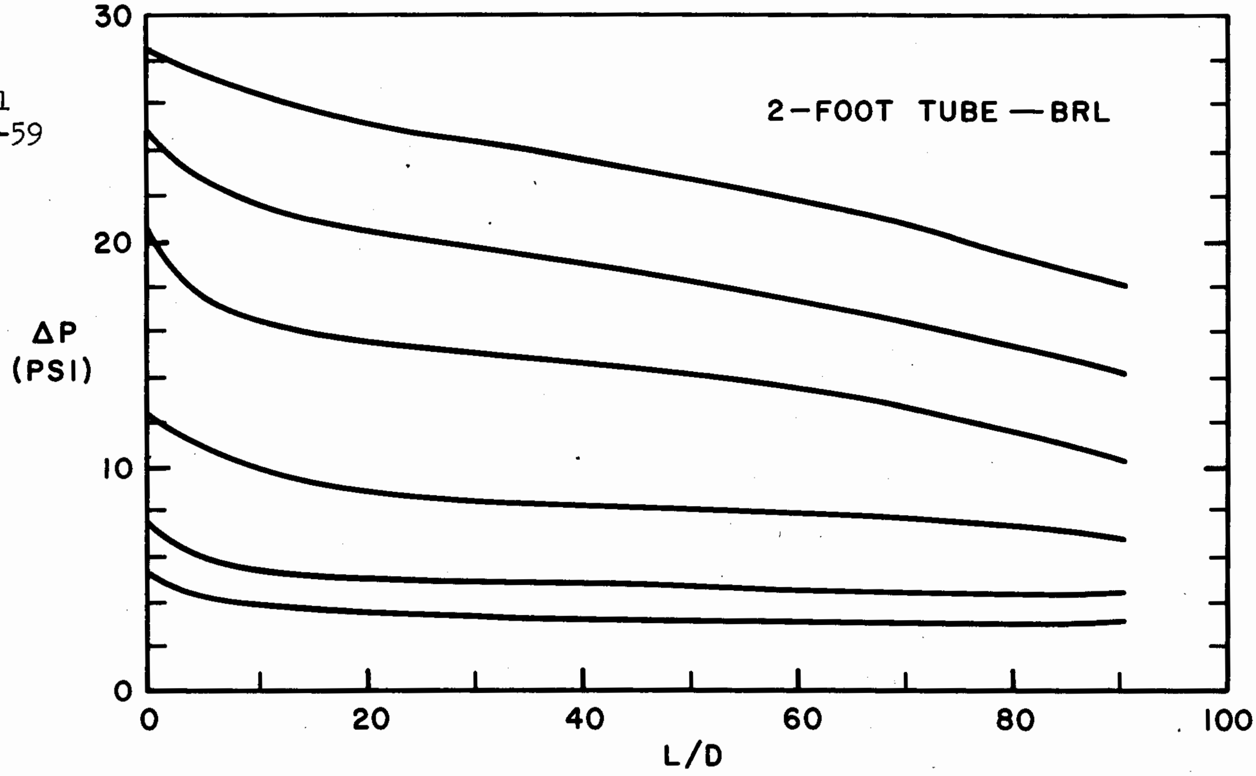

Fig. 3 — Shock tube attenuation data

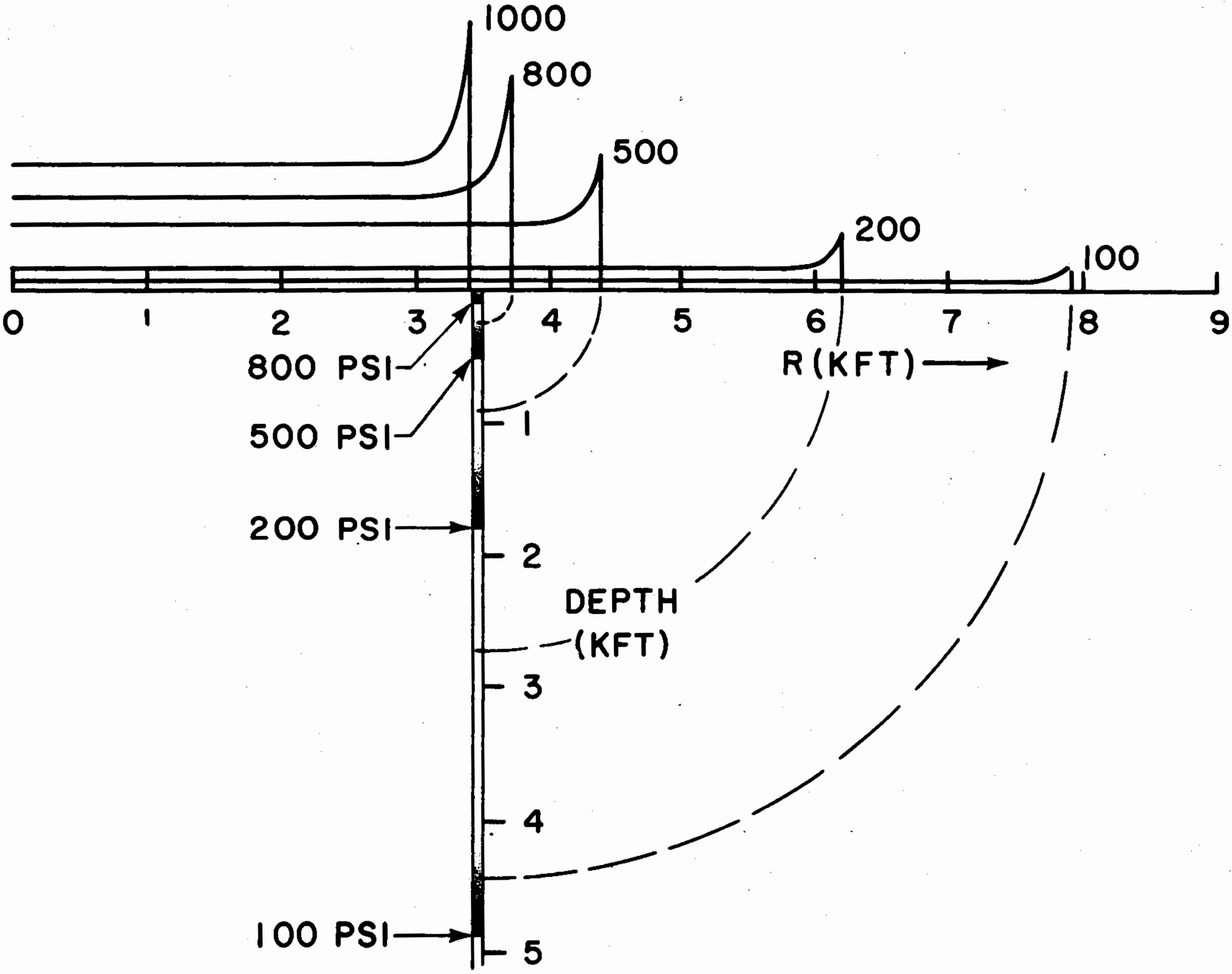

Fig. 4 — 10 MT surface burst
(Tunnel portal 1000 PSI)

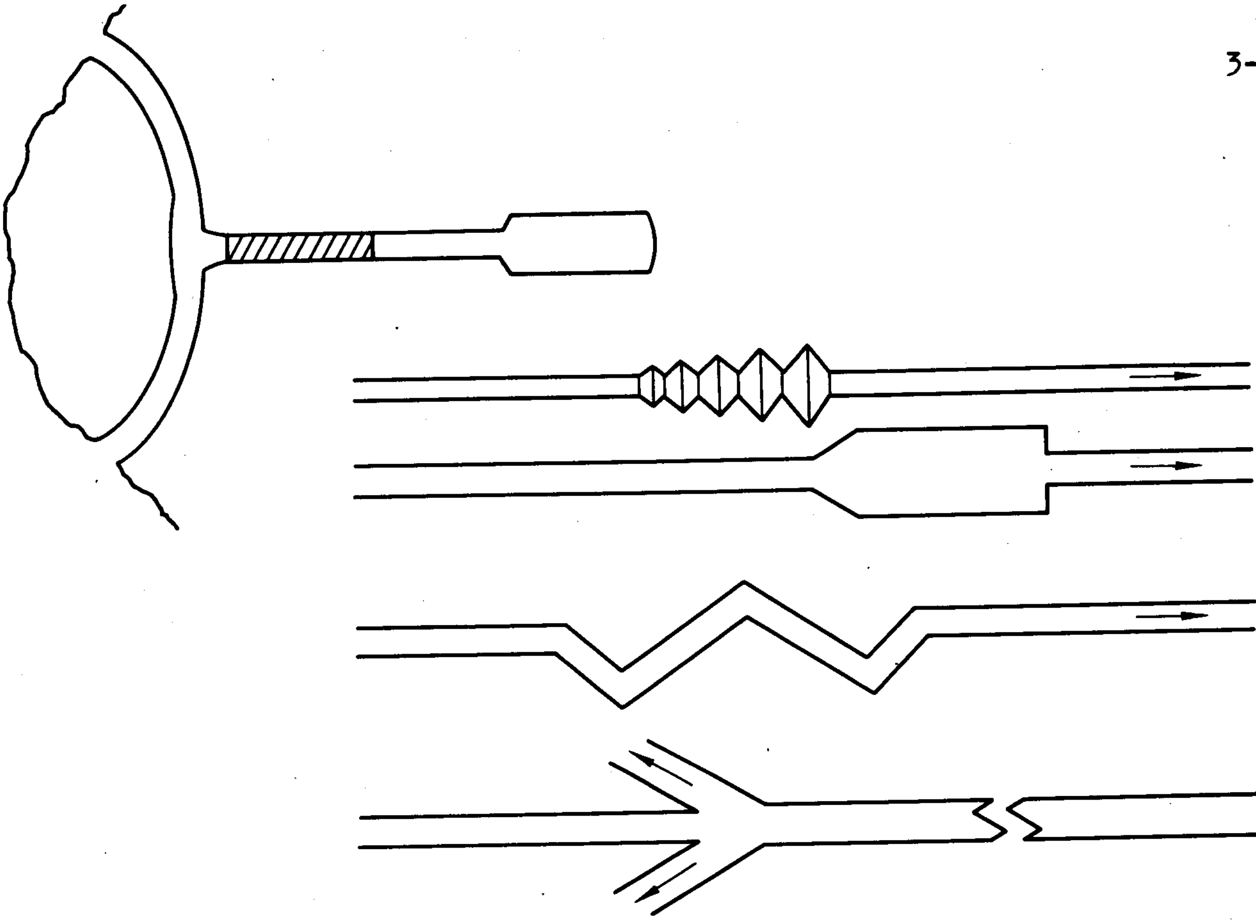

Fig. 5 — Shock attenuating schemes

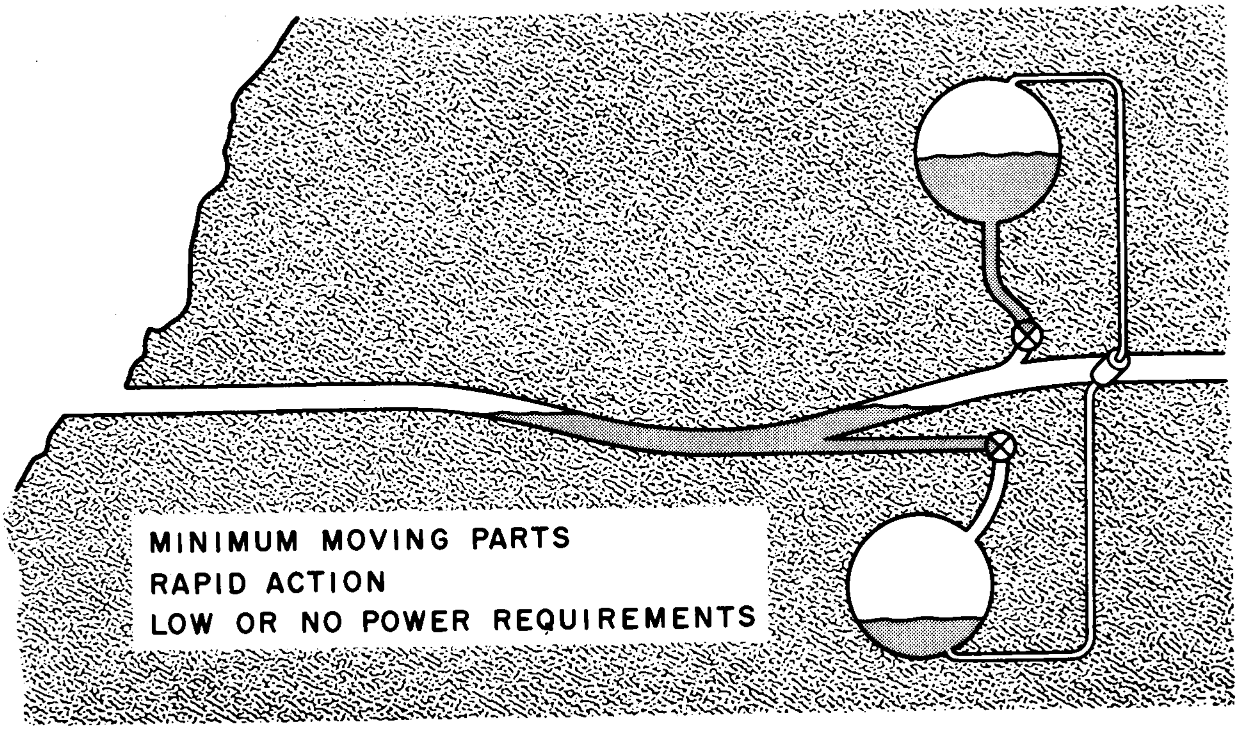

Fig. 6 — Water door

ANTENNA HARDENING

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INTRODUCTION

To have defense installations which are hardened against attacks by nuclear weapons and are capable of retaliation often depends on the adequacy of the communications system to resist these attacks. Without a hardened communication system, the necessary contact with command posts and coordination with other bases becomes impossible. The antenna is generally the weak link in the hardened radio-voice communication system because it is inherently an exposed structure when in operation and is difficult to protect. Similarly, antennas for missile guidance and radar detection are equally as important and are equally as vulnerable.

This presentation is confined to the structural hardening of antenna systems. However, to insure an efficient and economical system, a close liaison is necessary in the design between the structural engineers and the electronics engineers. Since structural considerations often conflict with the most efficient electronic design and vice versa, the close coordination will lead to a practical design which is satisfactory both electronically and structurally.

Several possible means of providing hardened antennas are being studied at the present time. Some of these methods will be presented and briefly discussed. It must be remembered, however, that the method of hardening used depends on the adaptability of the particular antenna to the method, the degree of hardness required, equipment and materials available, whether sacrifices in antenna efficiency can be tolerated, and whether the particular method is economically feasible. Some methods have distinct advantages but may also have inherent limitations which make them impossible to adapt to a particular situation. For example, exposed towers make the most efficient antennas but at incident blast pressures exceeding about 10 psi, it becomes difficult or, in some cases, impossible to obtain a feasible structure to resist the loads.

The methods of providing hardened antenna systems which are being studied are:

(a) A hardening of exposed structures of existing types by modifying present designs.

(b) Use of new concepts and ideas in antenna design which are more readily hardenable.

(c) Using antennas which are sheltered from nuclear effects except when in operation; e.g., retractable antennas.

(d) Making use of existing terrain and hardened structures to support the antennas.

To illustrate the hardening of exposed structures, a recent study into the feasibility of hardening the 4-30 Mc Steerable Wullenweber Receiving Antennas has disclosed that the present system could be hardened to resist an incident blast pressure of 10 psi with a reasonable increase in cost. The antenna is an extensive system consisting of two parts, the outer 4-11 Mc system which is guyed and the inner 11-30 Mc which is a free-standing trussed cylinder as shown in Figs. 1 and 2. The blast loads associated with the 10-psi incident pressure could be resisted by increasing the size of the structural members. However, because of the nature of blast loadings, if the design pressure were to exceed 10 psi, the loads on the structure would increase considerably more than just a simple proportion. This will reflect itself in a similar increase in member sizes and cost of the structure. The hardening of exposed structures will be covered in more detail in the section on the design of hardened towers.

The same antenna system was studied using a second possibility of encasing the electronic system in a plastic foam type material. The configuration of the scheme as shown in Fig. 3 is a cylindrical concrete core having an outside radius of about 77 ft.; surrounding the concrete core is the torus-shaped foam plastic mass in which the antenna screens are embedded. The foam materials considered were those in the Urithane class. These are rigid foams which can be cast in place with the use of the most simple type equipment. These materials incorporate a relatively high degree of strength along with low thermal conductivity, resistance to vermin and water vapors, light weight and are infusible once they have set. However, the Urithanes are combustible and are relatively high in cost.

The study showed that with the use of this scheme, it is possible to design the antenna to resist incident shock pressures of from 25 psi to 350 psi depending on the density of foam used. However, the scheme is unpractical because of the high cost of the foam, the cost being about 50 times that of the conventional steel supporting structure. For this reason, further studies on the effects of thermal radiation and nuclear radiation on the foam were not conducted although it seems that these effects can be overcome.

The use of the foam encasement illustrates a new idea in antenna design which leads to a more readily hardenable antenna although in this case it turns out that the idea is unfeasible for economic reasons. Other new concepts which are being proposed and studied by communications engineers are based on the idea of partially avoiding blast effects by designing the antennas as low silhouettes close to or flush with the surface of the ground. Some of the antennas are of the lens type, dome shaped structures which can be controlled to radiate in a number of directions over a limited range; others are antennas which are mounted nearly flush with the ground and have similar limitations as to direction of radiation. Although these antennas are more readily adaptable to structural hardening, they suffer in efficiency of operation because of inherent limitations.

At the present time, sheltered antennas appear to be the most suitable method of providing hardenable systems. One type being used by the Air Force in the "Titan" bases now under construction (4)* is a retractable or "telescoping" system for missile guidance as shown in Fig. 4. Here the antenna is sheltered in a buried concrete cylinder which is designed to resist the ground shock. The antenna is kept in the down position until it is ready for operation so that it is protected from nuclear

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* Figures in parentheses indicate references listed at the end of the paper.

effects. When the antenna is needed, it is elevated into operating position; however, in this position, it is extremely vulnerable to nuclear attacks. When protection against multiple attacks is required standby antennas should be provided. In the figure, the "telescoping" antennas may be used as alternates in case the exposed tower is destroyed.

Other antennas of the sheltered type are also under consideration. Similar to the "telescoping" antenna is an antenna of the "tilt-up" type. This antenna is kept in a horizontal position below ground until it is ready for use as shown in Fig. 5. The roll away door is necessary to protect the antenna from nuclear effects. The antenna may be used in a manner similar to the "telescoping" system.

Another of the sheltered antennas is the encased system shown in Fig. 5. This is an above-ground antenna encased in a fold-away steel and concrete structure. When ready for operation, the encasing structure folds away in "orange peel" fashion leaving the antenna exposed. In the operating position, the antenna is vulnerable to attack and provision must be made to keep other antennas on standby to be used when required.

The fourth method, using built-in antennas, is advantageous where natural terrain or hardened structures exist which can be used for supporting the antennas as shown in Fig. 4. By mounting the antennas in this fashion, the antenna can be made almost as hard as the supporting medium. The method, however, is generally limited to antennas which are required to radiate only in one direction as shown in the figure.

Alignment is important for many types of antennas and therefore it is necessary that the antennas be shock mounted to limit any misalignment to such magnitude that may be readjusted readily. Automatic leveling and aligning devices may be installed in the systems for this purpose.

The four methods mentioned can provide antennas which are hardened within certain limits. For instance, exposed structures of present types may be hardenable to about 50 psi and sheltered structures may be hardenable to about 200 psi. In actual installations, especially with deep underground bases, to design the exposed components or even the sheltered subsurface components to a hardness equal to that being considered for the deeply buried components becomes impossible. For this reason and for other practical considerations such as economy, protection against multiple attacks, and antenna efficiency, we are led to use multiple systems consisting of several dispersed antennas of reasonably attainable hardness.

Free-standing above-ground antenna structures are generally the most efficient type. Protected systems and retractable systems are inherently limited in application and may often be less efficient. However, to design the exposed structures to resist the same weapons effects as the other components of a deeply buried installation becomes impossible.

One possible solution is to use a system employing alternate antennas. The primary antenna may be an exposed tower designed to resist a blast loading somewhat less than the loads for which the other components of the installation are designed. The alternate antenna may be of the retractable type. If a blast occurs which destroys the exposed structure, the telescoping antenna, which has been in the protected position up to this time, is then put into operation. Although this antenna may be less efficient, it will be adequate for emergency use. To protect against the possibility of multiple attacks it is necessary to install standby protected antennas.

A second solution is to use a number of hardened structures and to disperse them in such a way that there will always be one or more antennas which are operable after an attack.

The greater the number of antennas and the greater the dispersal between them, the lower the design pressure may be for the antennas. However, increases in the number of towers and distance between them also increases the cost of ground lines and boosters required to feed the antennas from the base installation. Factors of economy such as drawing a balance between line costs to the installation and antenna costs, considerations of terrain features, protection of lines from the blast effects, and many other considerations all enter into determining a dispersed system.

Here also, it may be necessary to make provision for protection against multiple attacks. However, where several dispersed antennas are used, protection against multiple attacks is automatically provided since several shots are required before all the towers are destroyed.

The preceding discussions have served to illustrate the methods in which hardened antennas and antenna systems may be obtained. The remainder of the presentation will be concerned with the hardening of exposed structures and a discussion of blast effects as they pertain to antenna structures.

DESIGN OF HARDENED TOWERS

The first step in the design of any structure is to determine the loads acting on it. The loads on a structure associated with a nuclear explosion are the result of the incident and reflected pressures from the shock front striking the structure and the drag forces due to the high velocity winds which follow directly behind the shock front (5). In tower structures where the members are small in size, the time required for the blast wave to surround the members is so small, in the order of miliseconds, that the structure does not have time to respond to the load and build up stresses. Therefore, the incident and reflected pressures can be neglected often and only the dynamic pressures due to wind forces need be considered.

Figure 6 shows a typical load vs. time curve for loads being applied to a trussed tower. The peaks in the curve indicate the reflected pressures and it is seen that the duration of these loads is extremely short. The rise in the curve is due to the build-up of wind loads on the tower as the wind passes the front face of the tower and strikes the side faces. It is clear that the short duration of the shock loads does not allow the time required for the structures to build up stresses under this load and therefore neglecting these loads is completely justified.

Neglecting shock loads does not apply, however, where the size of the members are large or where the members are stiff enough so that they respond very quickly to these loads. In this case, the stresses due to shock load may be very important and must be taken into account.

Figure 7 shows several values of peak dynamic pressure due to the wind associated with the values of incident overpressure of 5 psi, 10 psi, 15 psi and 20 psi. In contrast to the values shown, towers designed to resist winds of 124 MPH are designed for a static pressure of about 0.50 psi ultimate strength. The maximum value of wind velocity for which "conventional" structures have been designed is 200 MPH, corresponding to a static pressure of less than 1 psi. It is clear that "conventional" structures could not resist blast loads even at the lower pressure levels.

Also listed in the figure are the clearing times for the incident shock front. However, even for the relatively large 3 ft. member, the clearing time is very small compared to the duration of the dynamic pressures which can vary from about 0.4 sec. for a small bomb in the kiloton range to about 4 sec. or longer for a large bomb in the megaton range. However, for larger members, the duration of the shock wave may become large enough to be important.

Figure 8 shows the comparison between the computed pressures and the observed pressures for a 43 KT bomb at a height of 700 ft. as obtained from tests at Operation PLUMBBOB (12, 13). There is considerable discrepancy between the computed dynamic pressures and the observed dynamic pressures although the values of incident shock pressure agree fairly well. The discrepancies are due to the presence of precursors in the shock wave (5) which increase the dynamic pressures above those which are theoretically expected. Since tower structures are sensitive to these pressures, it is necessary to make some estimate of the increase in pressure by precursors in order to insure a safe design.

The duration of load is important in the design of structures since it determines the response of the structures to the load. Blast loads are assumed to be instantaneously applied dynamic loads which decay exponentially with time. Figure 9 shows the shape of the idealized function. In actuality, there is some time of rise and irregularities in the curves, especially if the shot is not completely unobstructed. For ease of computation, the exponential loading curve is approximated by the triangular loading shown. The curve shows that the effective duration of dynamic pressures, td, can be much less than the effective duration of shock loads, T. Where the duration of load is long, with respect to the natural period of the structure, the load can be further simplified by assuming it as constant with time. It should be noted that the effective duration of the drag load is the time required for the pressures to decay to a zero value in contrast to the clearing time for the shock loads.

The effect of ground shock on electronic equipment is important and must also be considered. The equipment must be shock mounted to prevent damage during an attack. The type of shock mounting employed depends on the frequencies of the ground shock and of the equipment being protected.

The drag forces on structures due to the winds accompanying nuclear explosions differ from the forces on the structure due to ordinary winds because the velocity of the winds is generally so high that compressibility of the air, which is normally neglected, must be taken into account.

However, where incident blast pressures are less than 10 psi, the wind velocities are such that they are in the range where the effects of compressibility can be neglected and the forces may be obtained by using the same drag coefficients that apply to structures under ordinary winds (5).

Drag coefficients at the lower wind velocities are functions of Reynolds number as shown in Fig. 10. The figure shows that the drag on flat plates is practically independent of Reynolds number except at the lower values. Similarly, the drag on all flat members such as I-beams, angles, channels, etc., is independent of Reynolds number.

However, cylindrical members are sensitive to Reynolds number except at the very high values. The difference in the curves for the infinitely long cylinder and the cylinder having a ratio of length to width of 5:1 indicates the effect of slenderness on the drag coefficients. At the higher Reynolds number, even though experimental data is lacking, there is no reason to suspect that the values will vary much. The curves shown in Fig. 10 are for members with smooth surfaces; for rough surfaces, the drag coefficients must be increased.

Drag coefficients for trussed structures may be presented as a function of the solidity ratio, the ratio of the solid area divided by the total enclosed area, and are dependent on the shapes of the members (1). Furthermore, trussed towers are composed of several plane trusses which are placed in such a manner that the forces on the leeward trusses are reduced by the shielding effect of the windward trusses. The amount of shielding is dependent on the solidity ratio and the shapes of the members, whether they are flat or round.

Figure 11 shows the composite shape factors for trussed towers of square cross section and triangular cross section with wind blowing normal to a face as a function of solidity ratio (1). The curves take into account the effects of shielding on the leeward trusses. It is seen that the curves conform very closely to the test data available. For towers composed of round members, the curves shown were obtained by taking 2/3 the values of the coefficients for towers with flat members. These values are apparently conservative from the limited test data available. For towers composed of a combination of flat members and round members, an interpolation based on a weighted average of flat areas to round seems to be a reasonable approach.

The angle which the tower makes with the direction of wind also has an important effect on the drag coefficients for towers. For instance, the maximum load on square towers occurs with the wind blowing along a diagonal and with triangular towers with wind blowing normal to a face as indicated in Figure 12. The figure shows the results of tests on towers to determine the ratio of drag coefficients for a yaw of ψ° to that for wind blowing normal to a face. For square towers, it is shown that the ratio is always greater than one, indicating that the lowest value of drag occurs with wind blowing normal to a face. The line is a conservative recommendation for the ratio for a 45° yaw on a square tower. For triangular towers, it is shown that the ratio is always less than one, indicating that maximum drag occurs with wind blowing normal to a face.

In the transonic and supersonic range of wind velocities, the effects of compressible flow, which are negligible at the lower velocities, become controlling. Drag forces in the higher velocities become functions of Mach number, the ratio of the air particle velocity to the velocity of sound.

At Mach numbers of less than 0.4, the preceding discussion for subsonic wind velocities applies. However, when the peak incident overpressure exceeds 10 psi, the air particle velocities are such that the Mach number is greater than 0.4 and the drag coefficients for transonic and supersonic wind velocities must be used. However, even at the higher pressure levels, the Mach numbers may start out at high values but because of the decaying nature of the winds associated with a nuclear explosion, the Mach numbers will also decrease with time to values which are in the subsonic range. Therefore, it may be necessary to use supersonic, transonic and subsonic drag coefficients if a complete loading history is desired.

Information available for drag coefficients at the higher Mach numbers is limited to that for cylinders (8, 9). For this reason, we will limit our discussion to that for cylindrical members only. Although the material available may be used in conjunction with sound engineering judgment to design structures to satisfy present requirements, additional data would be highly desirable. The information presented here illustrates the phenomena and problems which are encountered when designing structures to resist transonic and supersonic winds.

At Mach numbers above 0.4, compression waves form on the structure due to the compressibility of the air at these high velocities. Figure 13 shows the result of free flight tests on three different-size cylinders (9). The curves give some indication of the effect of Reynolds number and Mach number on the drag coefficients. Because Reynolds number is a function of the velocity, the values vary with Mach number. The spacing between the curves at a given Mach number indicates the effect of Reynolds number on the drag coefficients. It is seen that at the lower values of Mach number, Reynolds number has considerable effect on the values of drag coefficients.

As the Mach number increases above 0.4, the drag coefficients for the cylinders begin to increase rapidly until a value of Mach number of approximately 1.0 is reached. In this range, there is still considerable effect of Reynolds number. However, as the Mach number increases above 1.0, the effect of Reynolds number virtually disappears. The overall drag coefficients decrease as the Mach number increases above 1.0.

The major effects of Mach number on the drag coefficients for cylinders are confined to the range of 0.4 to 1.4. This conforms to particle velocities associated with incident blast pressures of from about 10 psi to about 100 psi. This is the range within which most hardened tower structures will be designed.

When analyzing for the loads associated with nuclear weapons, the duration of the load is as important as the forces themselves (5, 6, 7). The duration associated with a particular blast pressure is dependent on the size of the weapon causing the pressure and therefore it is important to know the characteristics of the weapon which you are protecting against, as was previously illustrated.

With tower structures, which in general have long natural periods of vibration, short duration loadings will be relatively ineffective on the structure, whereas long duration loadings of considerably less peak pressures can be more effective. Figure 14 shows the relationship of dynamic load factor versus the ratio of load duration to the natural period of vibration for the element for linearly elastic systems, structures in which the internal resistance increases linearly with deflection (6). The curve serves to show that the duration of the load can be as important in determining the maximum stresses as are the peak pressures.

For example, if we have a tower with a natural period of three seconds and a weapon with a duration of load of 1 sec., the ratio T/TN = 0.333 and the dynamic load factor from the chart is 0.90. However, if we have a weapon whose duration of load is 3 sec., the ratio of T/TN = 1.0 and the dynamic load factor from the chart is 1.7. In other words, the dynamic load factor for the long duration load is almost twice that for the short duration load and the stresses are proportionally increased. Similar curves for other types of loadings are also available (6).

It should be further noted that although the tower itself may have a long period of vibration, it is possible that the members composing the tower may have much shorter periods. Therefore, the dynamic load factors for the individual members may be considerably larger than those for the tower and the design stresses must be obtained separately.

Cantilever or free-standing type towers may be analyzed as linearly elastic systems. The analysis of this type of structure is relatively easy to perform and can be accomplished in a reasonable length of time. However, guyed towers are not linearly elastic systems and the analysis is both difficult and time consuming. These towers must be analyzed as beam columns on elastic supports using methods presented in references 2, 3, 6 and 7.

A final point on the analysis of towers that should be noted is that most of the commonly used methods take into account only the first mode of vibration of the structure. Although this may be sufficiently accurate in many instances, there are cases in which the results based on this assumption may be misleading. For a complete analysis of a structure under dynamic loads, several modes of vibration should be taken into account (10). The more modes used, the more accurate is the solution.

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DISCUSSION

MR. CHARLES SANDOVAL (RAND Corporation): Pat, I have two questions. First, what would be the effect on the foam of other weapon effects such as thermal radiation, neutrons, earth shock, and this sort of thing?

MR. DI NAPOLI: As far as the nuclear radiation effects, such as gamma rays, are concerned, I have no data available. As far as thermal effects are concerned, these foams are flammable but they are also excellent insulators.

Now, the foams can be protected against continued burning by using some sort of filler. Tests have been made on the subject, and it has been found that by using a filler such as asbestos the foam can be made flame-resistant and self-extinguishing. But when you get close to ground zero, the thermal radiation is so intense that you would get considerable burn-off of the outer layers of the foam. In this case, you would have to provide some skin to protect the foam against burn-off, or use a much larger cover on the over-all system which, if burned off, will not affect the operation of the system.

MR. SANDOVAL: Second, in your guyed towers, for example, is there any loading due to von Karman vortices? Has any of this been noted?

MR. DI NAPOLI: The answer is yes, especially where you have slender flexible members such as the cables in the structure. There is little difference between the von Karman vortex effects in the lower pressure levels (with the wind associated with the lower pressure levels of blast) and the normal wind you would have in a storm.

However, at the higher pressure levels where Mach number effects come in, this is questionable. No data has been found on the subject.

MR. S. S. GREENFIELD (Parsons, Brinkerhoff, Hall and McDonald, New York City, New York): We seem to be neglecting the reflected incident pressures because of the rapid clearing time. Isn't there considerable energy to be absorbed due to the impulse loading, the impulse on the members?

MR. DI NAPOLI: The impulse on the slender members does not come from the shock wave passing the structure because with the very short clearing times and the long natural period of these members the response of the members is usually very small. Similarly, the energy imparted to the structure from reflected overpressures is also very small because of the extremely short duration of these pressures on slender members.

However, the dynamic pressures are impulse loads and must be taken into account dynamically in the design. As was illustrated, the dynamic pressures are by far the predominant loads on lattice-type structures.

MR. E. R. KUMMERLE (Burns and Roe, New York City, New York): At very high pressures, how do you take into account the effect of ground shock on the stability?

MR. DI NAPOLI: The effect of ground shock on the stability is important in the design of the structures. Some sort of shock mounting has to be provided in the structure and especially for the electronic components.

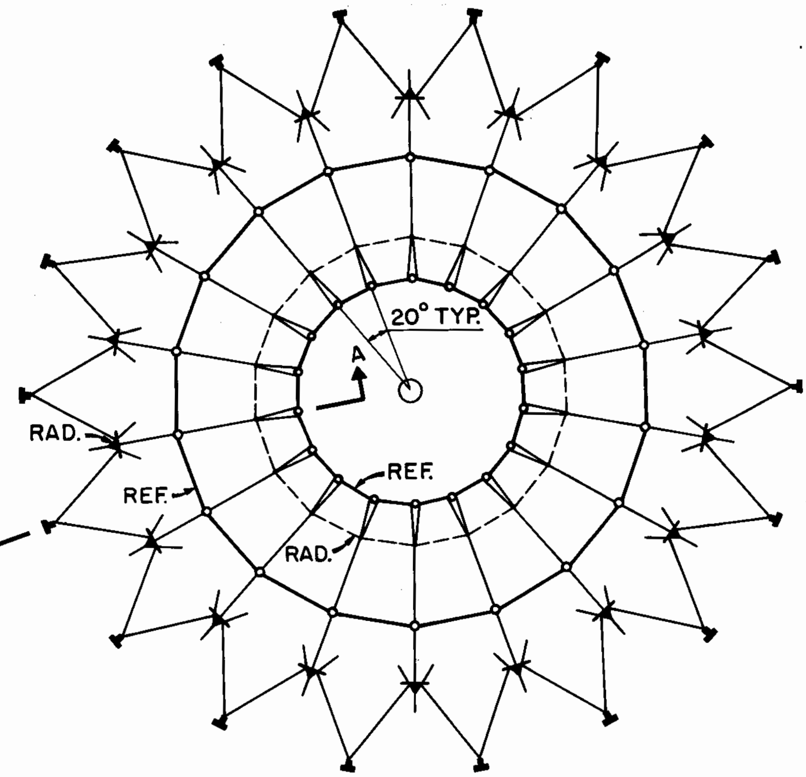
Now, there are several methods available for providing shock mounting depending on the size and frequency of the structure as well as the shock spectrum of the construction site.

However, in a guyed tower, you do get some shock protection due to the fact that the guys themselves are flexible shock mounts and will allow the structure to move without building up very high stresses. The long natural period of the structures is also a help in protecting against the sudden accelerations since the structure cannot respond to these accelerations.

MR. CHARLES KINGERY (Ballistic Research Laboratories, Aberdeen Proving Ground, Maryland): Do you advocate using foam to protect these towers? I'm afraid I don't understand how you are going to anchor the foam. There is a structure in it. Doesn't the foam have a tendency to move along with the dynamic pressure?

MR. DI NAPOLI: No, we do not advocate the use of the foam to protect these towers. The study was conducted on the basis of an idea and, as was mentioned, the foam was found to be too expensive to make the idea feasible.

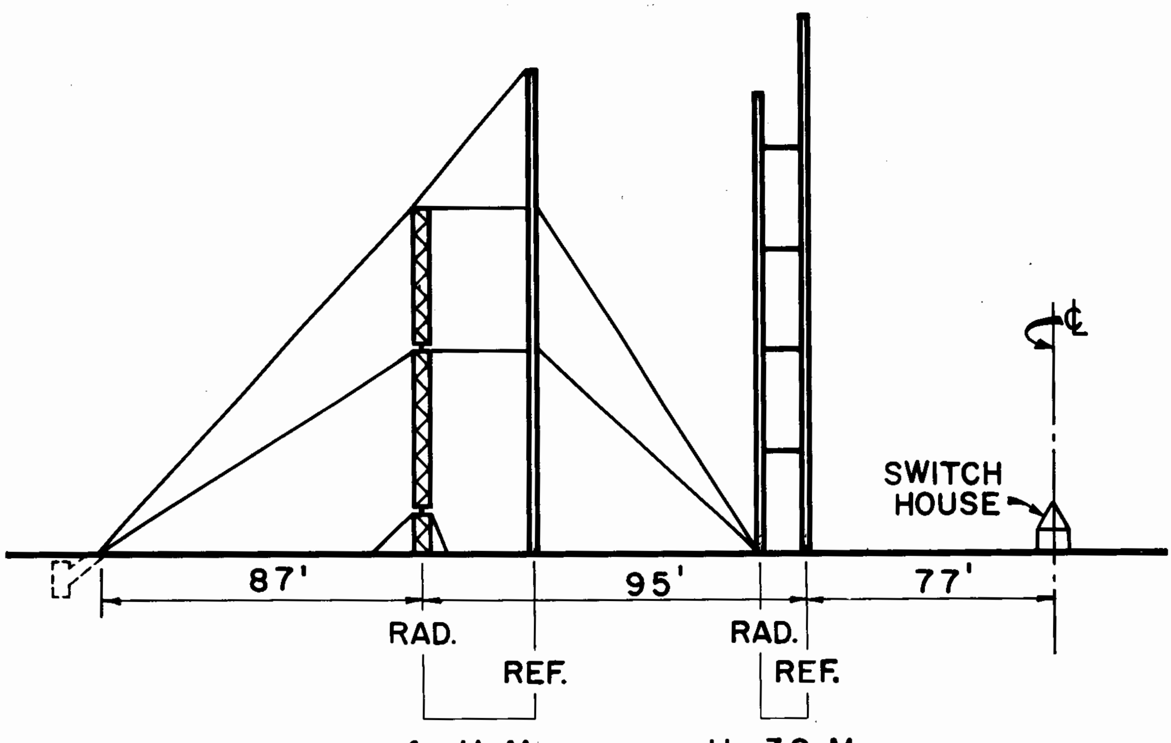
As far as the mechanics of the system are concerned, the concrete core is designed to take the transient blast loads. The foams are rigid foams which are cast in place and develop a high bond with the concrete core. These foams are capable of developing high strengths depending on the density of foams used. Because of these high strengths, the foam can resist any tendency to be moved along by the dynamic pressures and transmit the transient load to the concrete core.



Plan

4 - 30 MC ANTENNA

FIGURE 1



SECTION A-A

4 - 30 MC ANTENNA

FIGURE 2

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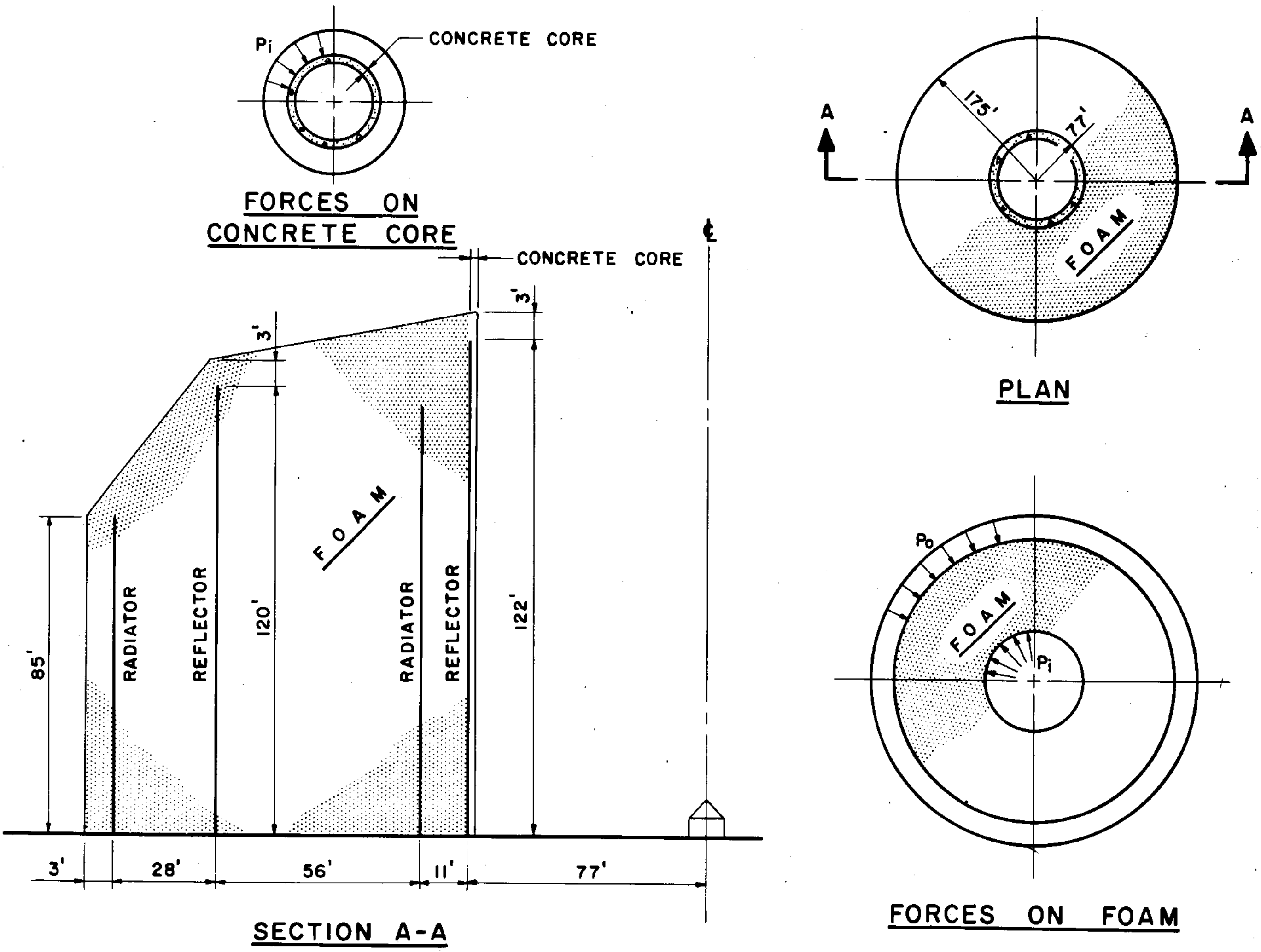
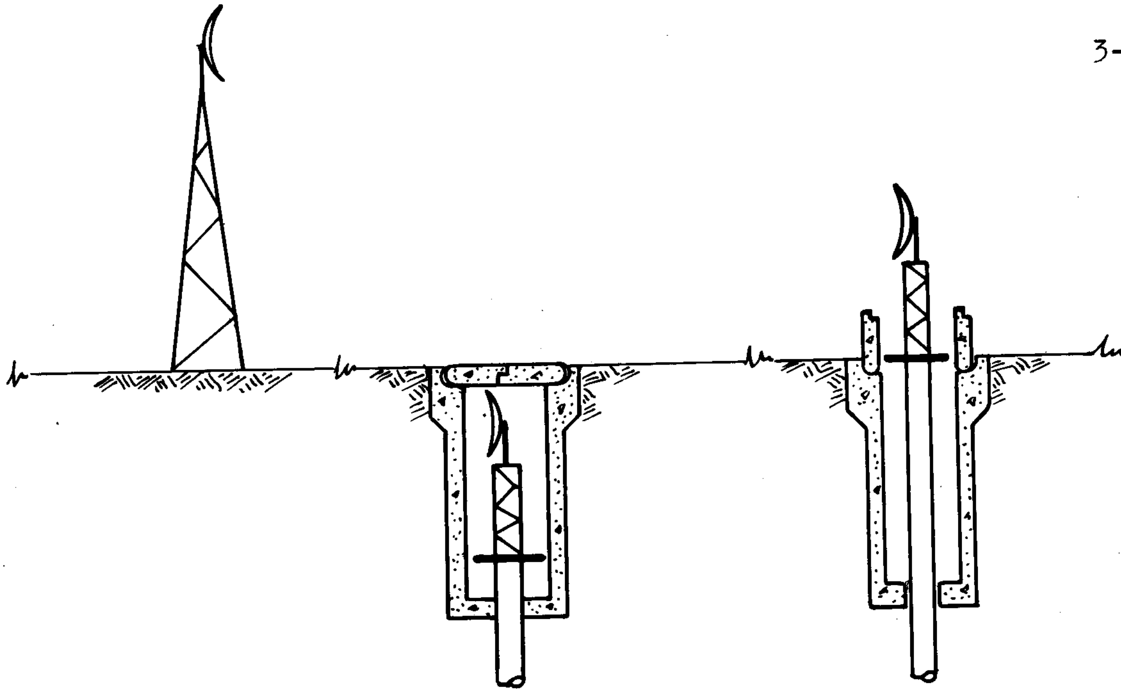
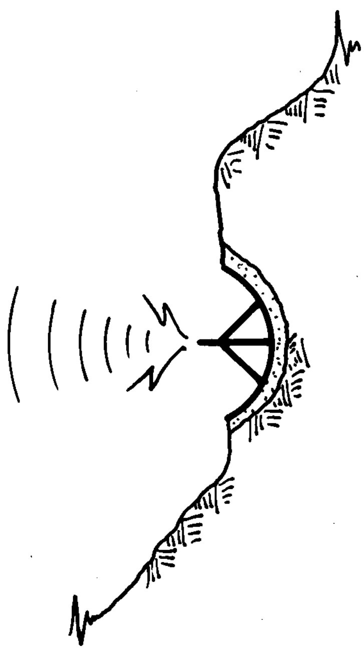
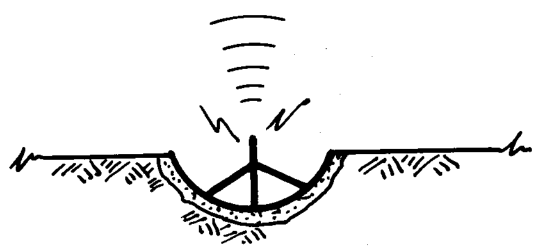


FIGURE 3. General Configuration of Antenna Encasement and Forces Acting on the System



TELESCOPING SYSTEM





VERTICAL RADIATOR

HORIZONTAL RADIATOR

BUILT-IN SYSTEMS

FIGURE 4

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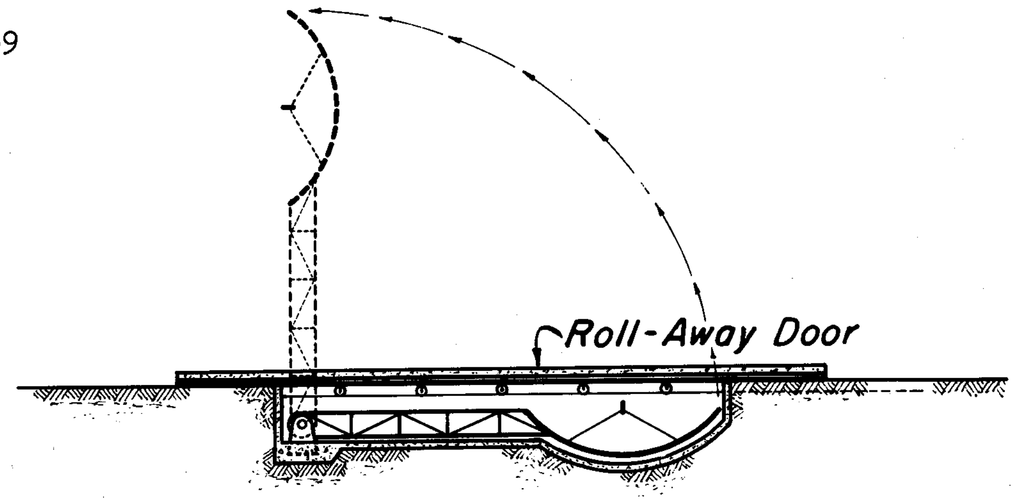
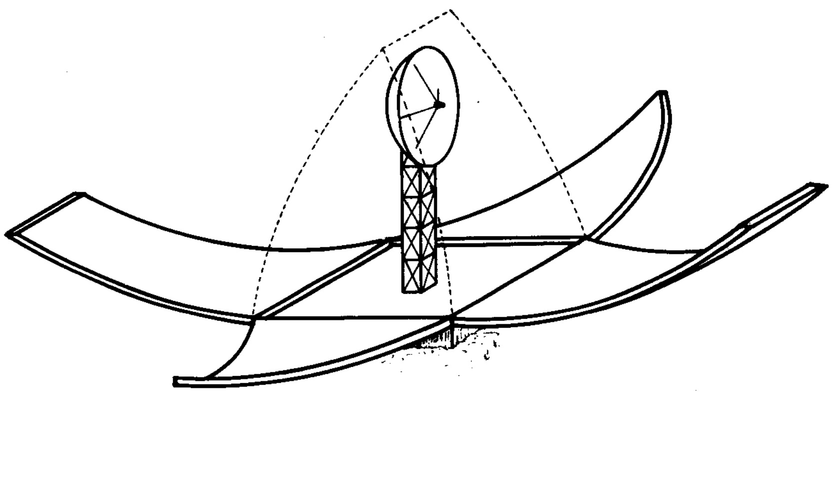


TILT-UP ANTENNA



FOLD-AWAY SHELTER

FIGURE 5

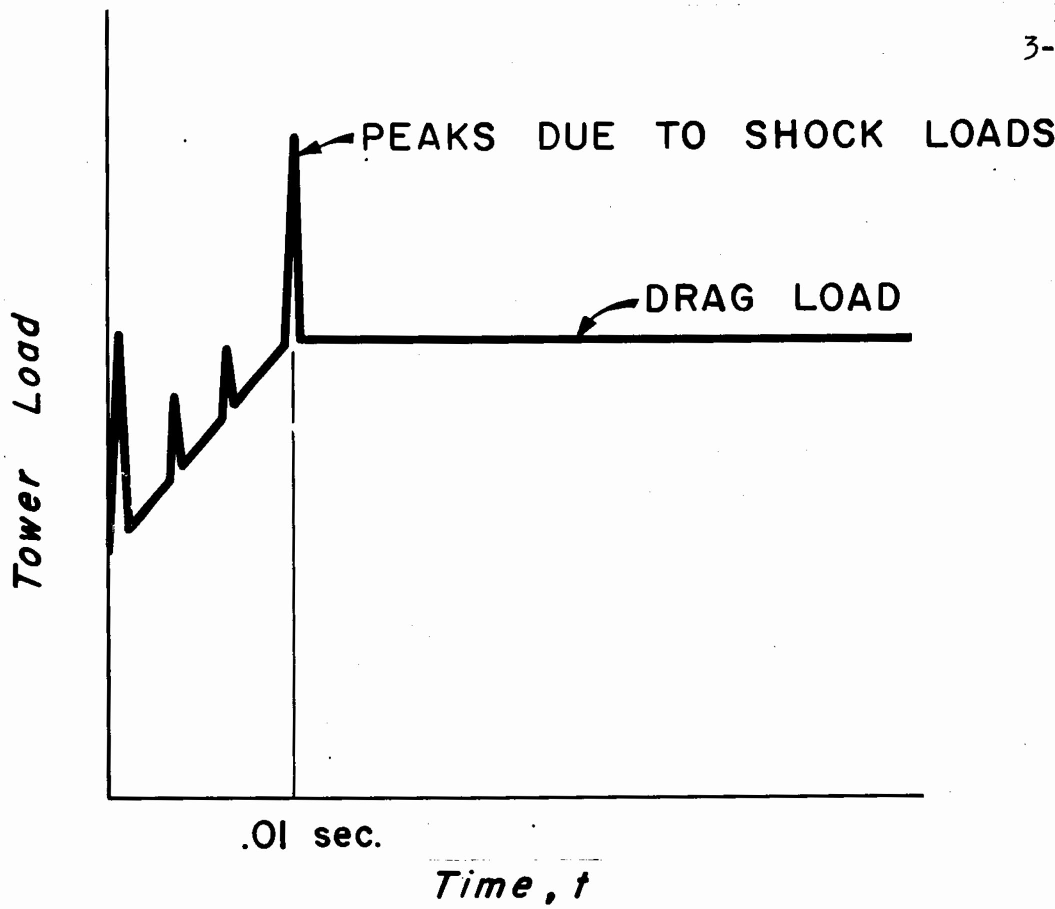


Fig. 6—Variation of tower load with time

PEAK INCIDENT OVERPRESSURE P_{so} (psi)	THEORETICAL PEAK DYNAMIC PRESSURE q_o (psi)	CLEARING TIME FOR SHOCK FRONT (sec.)	
		3 in. MEMBER	3 ft. MEMBER
5	0.60	.0002	.0024
10	2.25	.00017	.0021
15	4.80	.00016	.0019
20	8.15	.00015	.0018

Fig. 7—Incident overpressures and peak dynamic pressures

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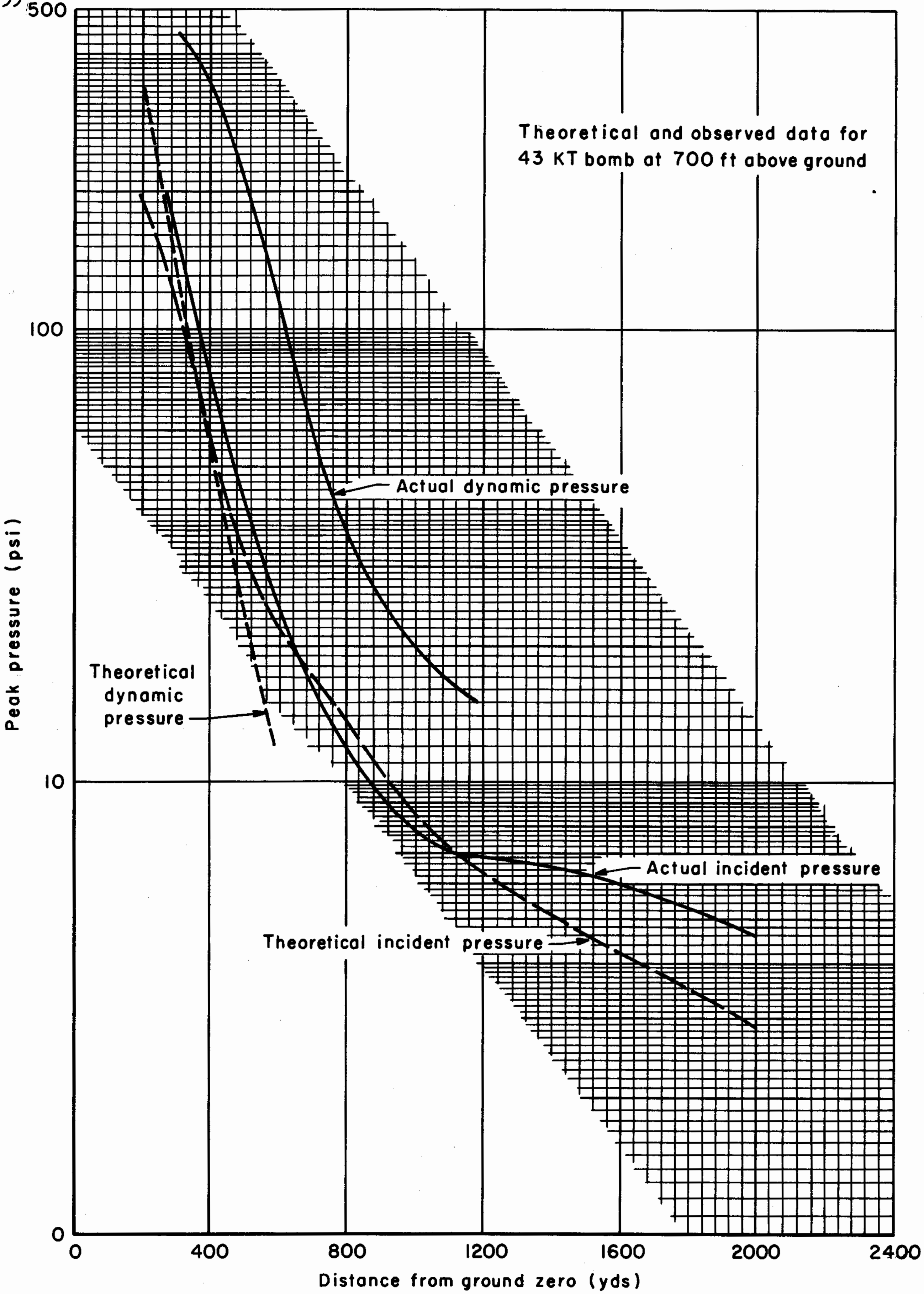
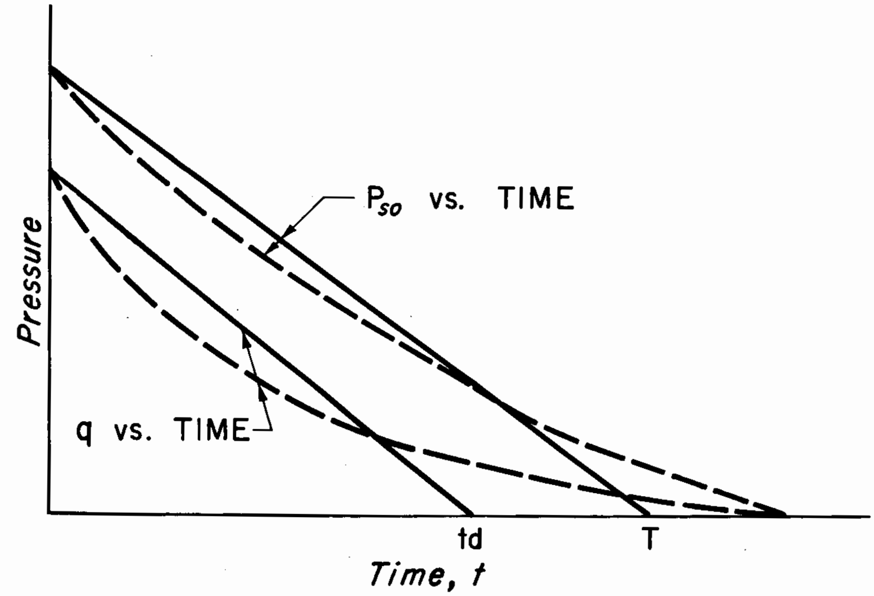
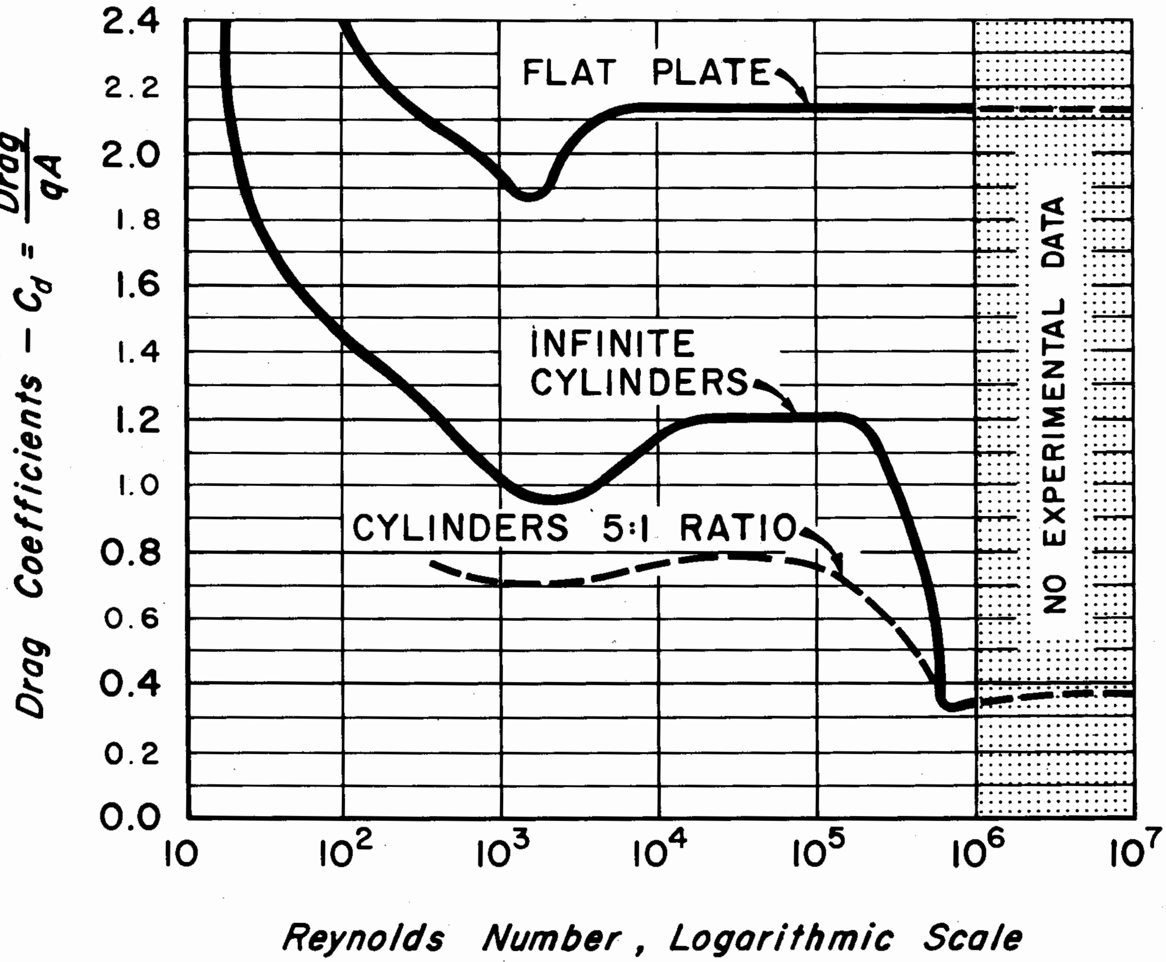


Fig. 8—Blast line pressure-distance curve



PRESSURE VERSUS TIME FOR DYNAMIC PRESSURE AND INCIDENT SHOCK PRESSURE

FIGURE 9



DRAG COEFFICIENTS FOR PLATES AND LONG CYLINDERS

FIGURE 10

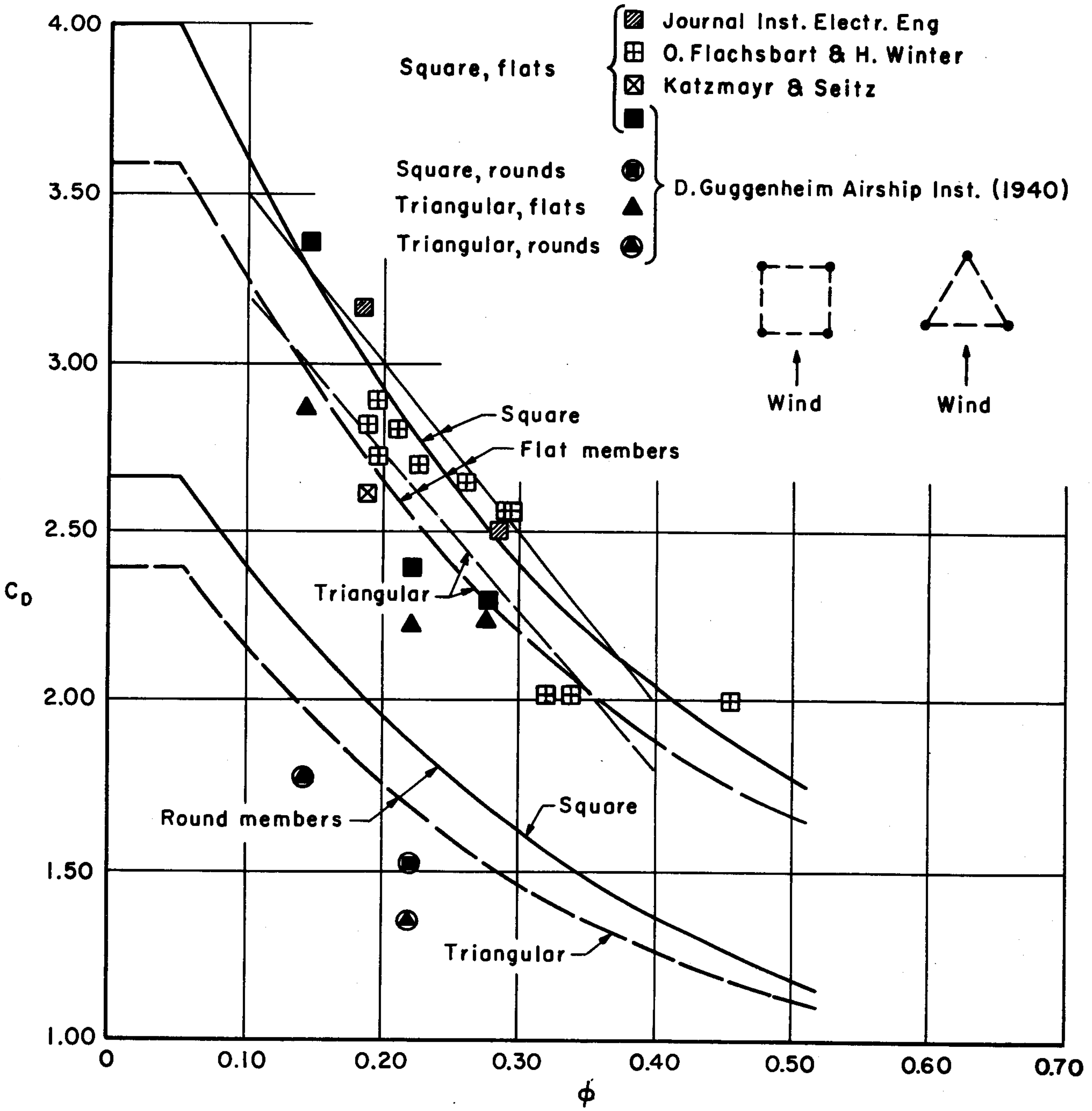


Fig. 11—Composite shape factors for trussed towers

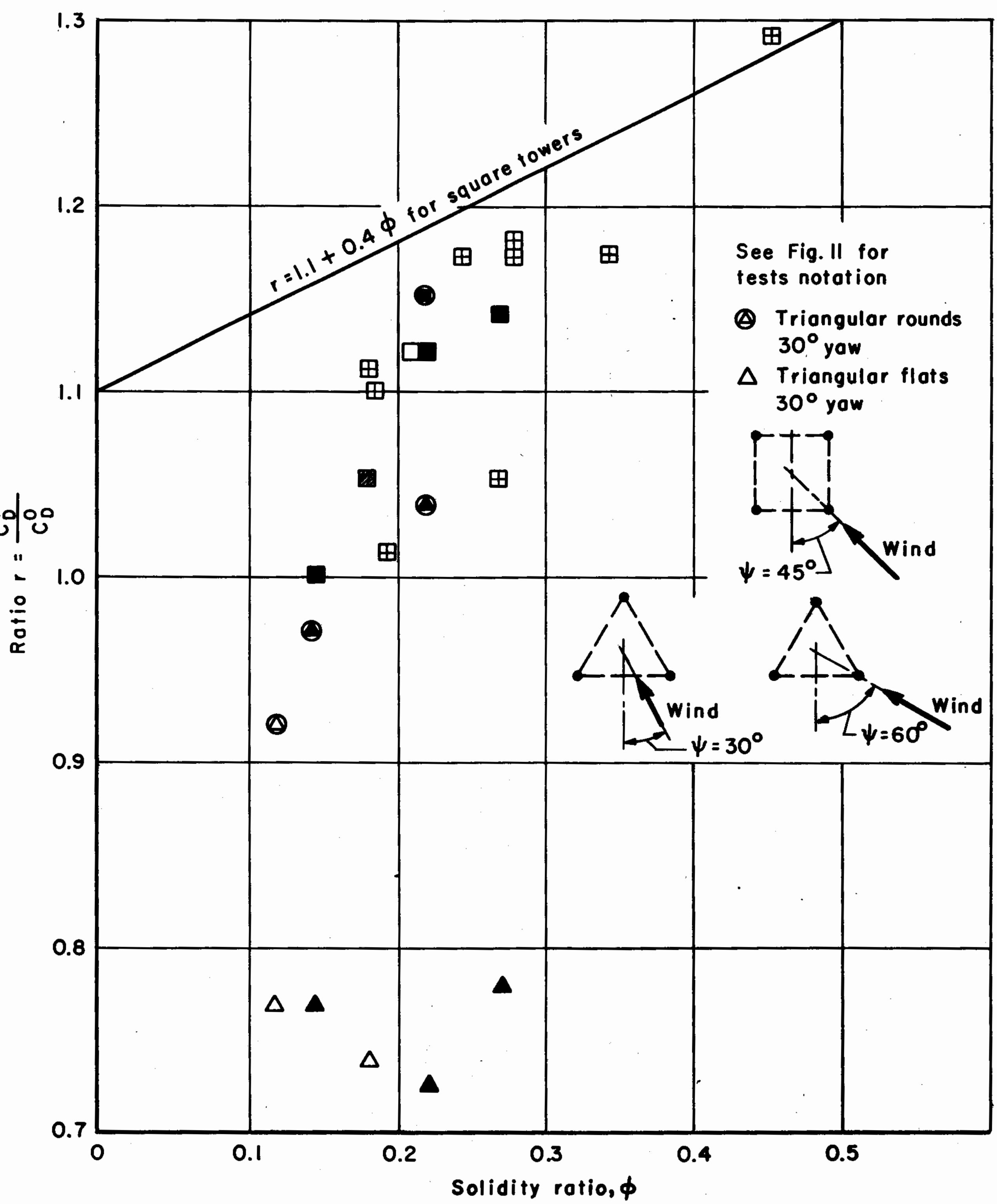


Fig. 12—Drag coefficient test results on towers

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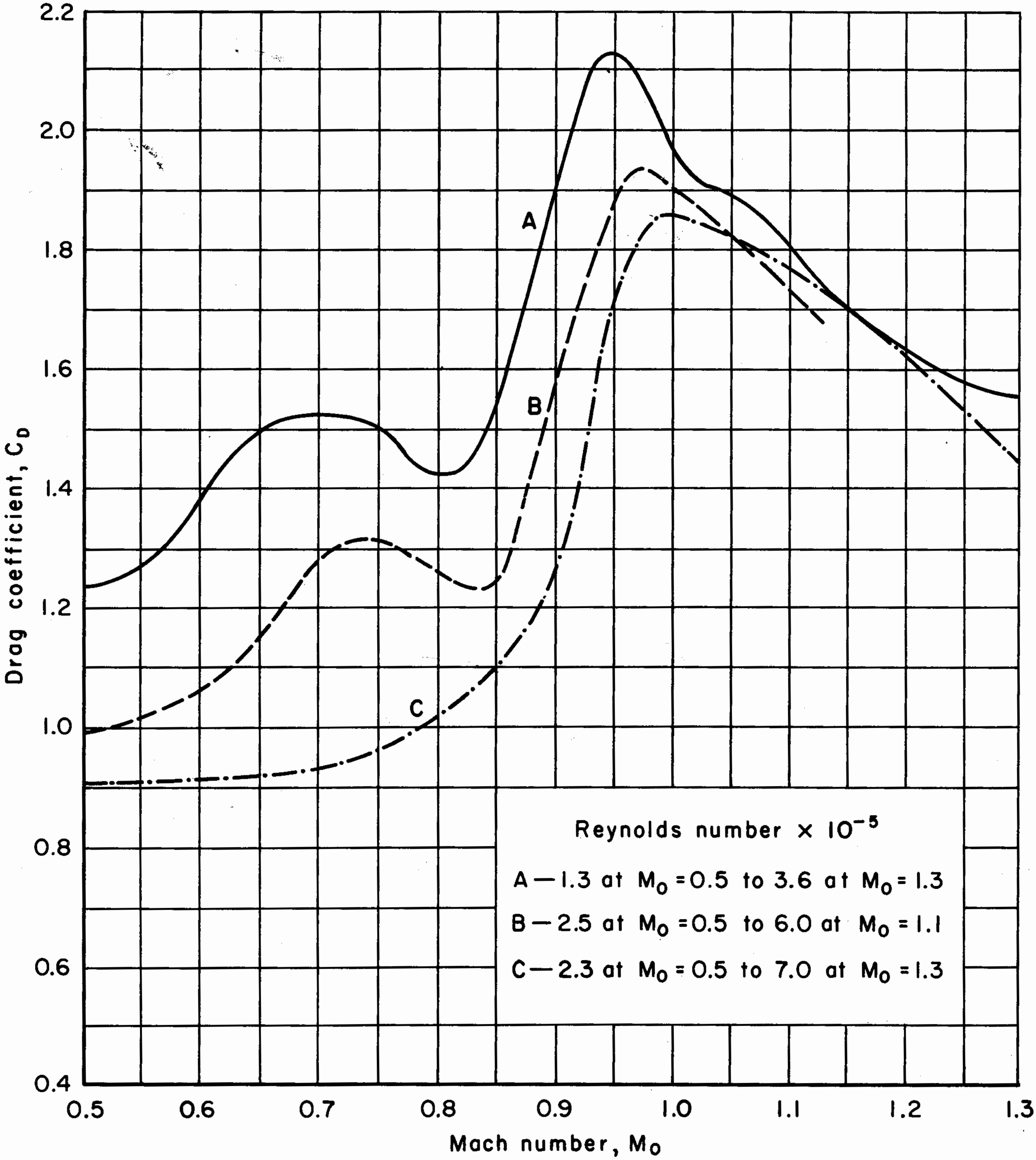
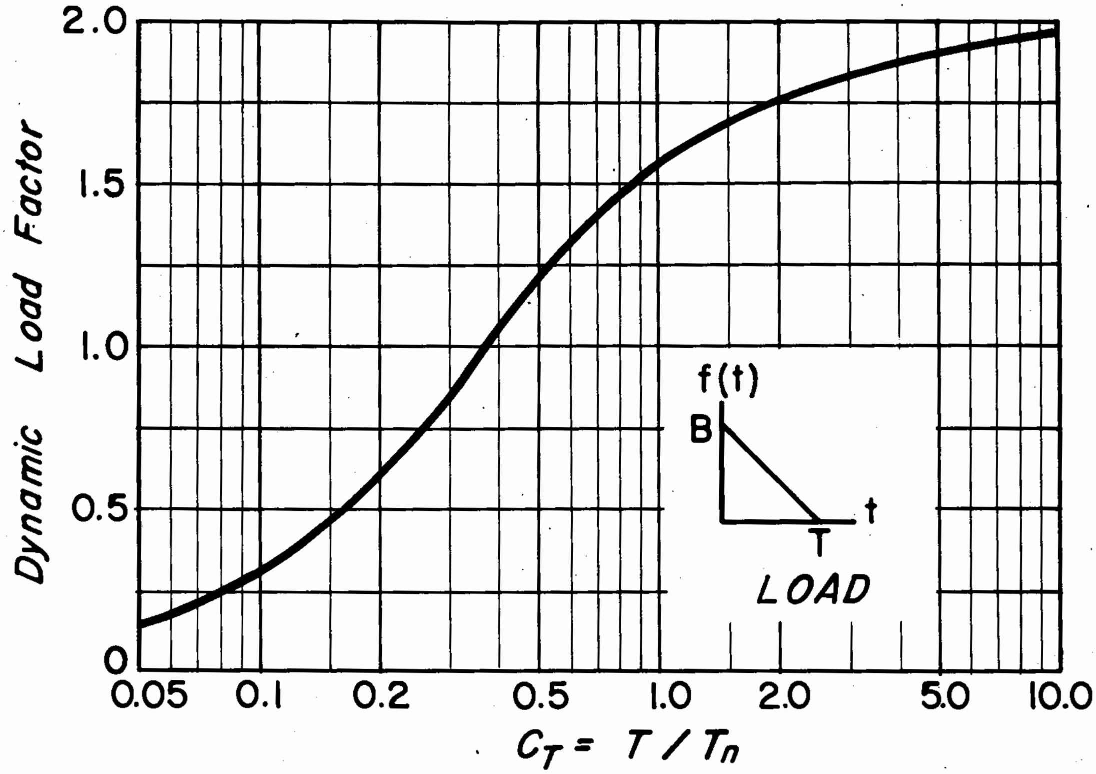


Fig. 13 — Variation of drag coefficient with Mach number for a circular cylinder



DYNAMIC LOAD FACTOR CURVE FOR LINEARLY ELASTIC SYSTEM TRIANGULAR LOAD

FIGURE 14

INTRODUCTION TO SECTION ON UTILITIES DESIGN

Bradley A. Peavy, Chairman
(National Bureau of Standards)

The purpose of this section is to present important utility problems to be faced in the design of deep underground military installations intended to resist the effects of nuclear weapons.

When we speak of utilities, we refer to the services and related equipment that provide air, water, electricity, air conditioning, and disposal of waste heat and sewage. All of the ground surface installations of the utilities that would supply the underground installation in peace-time for non-emergency conditions are to a greater or lesser degree subject to disruption by attack. The design of an underground installation must provide for all or most of these utilities within the protected area in anticipation of an emergency period.

In general, an installation is required to operate for perhaps some years during a normal, or peacetime, condition, but to be self-sustaining and operating after nuclear attack and fallout. It may be completely isolated from the rest of the world, except for communication channels, for a period of a month or less.

Under conditions of almost total severance from the outside world, the dependence of the utilities on each other becomes a vital link in the survival of the installation. This interrelationship of the utilities places new demands on the architectural and engineering design. The desired interrelationship, or interaction, of the utilities is fundamental and therefore will be the subject of two of the talks this afternoon. One talk will consider this problem for an underground installation, and the other talk will discuss the somewhat analogous situation of the submerged submarine.

By way of introduction, I will try to outline briefly a few more of the utility problems, most of which will be the subjects of the various talks. The problem of the disruption of the outside sources of electricity means that electrical generating equipment must be installed within the protected areas of the underground installations. The types of power-generating equipment suitable for use in underground installations will be the subject of one of the talks. Many attendant problems arise due to the need for air for combustion, water for cooling the equipment, and the exhaust for combustion products.

The success of the underground installation is dependent upon the ability of the air-conditioning system to maintain the atmosphere inside the area within certain limits of temperature, humidity, and purity. Suitable conditions for operation of equipment are a necessity.

The principal purpose of the air conditioner is to maintain suitable temperatures and humidities in spaces. A military facility has considerable electronic equipment for which the power demands are high. Most of the energy consumed by this equipment appears as heat (within the installation) which must be removed directly or indirectly by the air-conditioning system. The system must also absorb heat from such sources as light, personnel, electric motors, etc. These heat quantities, along with the power required to operate the system, must be disposed of in some manner.

For non-emergency use, water as a heat transfer medium is an answer to the problem. In an emergency condition it must be assumed that an external supply of cooling water may be interrupted and that adequate heat sinks must be provided for disposing of waste heat. If it were possible to choose a site with an underground river, most of the problems would be solved. However, it is hardly likely that such a condition would exist with other considerations for site selection. It is possible that seepage water may be used to supplement other methods of heat disposal.

In the general case, an underground water reservoir used to absorb the heat of the power-generating equipment is the solution to the problem. Later I will discuss the available heat sinks for underground installations.

Problems dealing with water for personal use are not as critical as those of the cooling water supply, but they demand that separate storage be made available and be protected against contamination of the outside water supply by covert or overt attack.

Air from the outside for combustion must be conveyed to the installation from the surface through tunnels or shafts a thousand feet or more in length and must be expelled from the installation by a separate exhaust system.

Air brought in for personnel must be purified by suitable filters because of possible contamination by chemical, biological, and radiological agents. If air for combustion must also be purified, the power required to purify the air is quite large, due to the high pressure drop across the purifying filters. Separate combustion air intakes of less filtering action may be considered.

The underground installation must be pressurized against infiltration of contaminants. Most installations must provide for ingress to the installation and attendant decontamination procedures. During an emergency condition, all outside services may be cut off. The fresh air supply may be cut off because of power failure or deliberately, if the locality has been excessively contaminated with radiological, biological, or chemical agents. Under this condition, the occupants may be forced to rely entirely on sources available within the installation for their respiration air, food, and water supply. This situation would be similar in many respects to that in a submerged submarine, except for the means of disposing of waste heat.

I have tried to touch on most of the utility problems to be encountered in underground installations. One that would be a major design problem but sometimes is not considered is the recovery of an installation to withstand a subsequent emergency period within days or a few weeks after the end of the initial emergency period. From the standpoint of disposing of stored waste heat, the recovery problem can represent enormous refrigeration loads. Restoration of the outside sources of utilities following the emergency period also deserves consideration.

THE INTERACTION OF UTILITIES

Jack W. Barrett and C.T. Tsitsera
Guy B. Panero Engineers

INTRODUCTION

The subject of this paper was first suggested at the conclusion of a study on the Manhattan Deep Shelter Project prepared by our firm in 1958. The scope of this study project, probably as ambitious as anything attempted to date, brought to light many interesting relationships between utilities which, although existing to some extent in conventional work, are not as apparent nor as important above ground as in the design of underground occupied spaces. This interaction of utilities, so called, may be defined as the relative effect, one on another, of each utility--HVAC, electrical, sanitary, and water supply--as the type and magnitude of each is varied in an attempt to reach a cumulative optimum within a required design criterion.

GENERAL

In the normal conventional above-grade building as shown in Fig. 1, we see the use of air for ventilation and combustion processes, and as a reservoir for heat rejected from the refrigeration cycle and other processes. These uses of air are taken for granted and little or no thought is given to the convenience of this cheap and inexhaustible combination heat sink, oxygen supply and exhaust disposal area.

In the above-ground design, the utilities are more or less independent and can be designed with no more than conventional consideration for their effect on each other. Air conditioning, for example, can be designed without too much reference to the other utilities. The number of people, floor area, type of construction, lighting and equipment loads, except for some services, are all known or can be estimated for design purposes. Equipment can be located in coordination with the architect to make air intake and exhaust runs short or esthetically acceptable. Several intakes and exhausts can be used. Cooling towers can be located outdoors on a roof or on grade. Vents for plumbing are located as required and there is no great problem in providing as many as required. Water supply is generally met by proper site selection, and storage can be provided with no major investment.

Power services are usually obtained from a local utility company.

Now let us move the installation underground with a degree of protection as indicated in Fig. 2. It is apparent that in order to utilize the atmosphere in the same sense in which it was used in the above-ground installation would require tremendous air intake and exhaust shaft areas at a severe penalty both in security and cost. For most major installations it is prohibitive to carry the atmospheric heat sink capabilities underground with the installation.

For installations where considerable protection is required, the shelter will involve hundreds of feet of cover with entrances of the order of thousands of feet long. There will probably be two access tunnels. It will be necessary to provide full power generation facilities, complete with required standbys. In many cases it will be advisable to operate the underground power plant continuously with possibly a standby tie line to the local utility.

Each of the underground utilities cannot now separately determine air intake and exhaust shaft requirements. Each separate major opening to the outside costs a considerable amount and the over-all design must be coordinated to keep the net area of these openings to a safe minimum.

For operation during periods when the installation is under attack, these openings are a potential weak link for the installation. Openings required for operation during this period should possibly be multiple small drilled openings, widely spaced and arranged to provide some degree of blast attenuation so that the probability of survival is increased. It is imperative that the outside connections be reduced to an absolute minimum, possibly to those required for combustion processes only.

In order to increase the protection of the installation, each of these openings may be provided with a blast closure device to automatically close when a blast occurs.

During normal operation of the underground space the heat which is rejected from the air conditioning system and from the power generators can be dissipated in the air by means of cooling towers located above ground. During button-up operations, however, the cooling towers are assumed to be inoperative and underground heat sinks utilizing water or ice as the heat absorber are required to store the waste heat.

It is necessary, when designing, to consider the costs of the installation to meet the two vastly different operating conditions--normal and "buttoned-up." The system should not be designed primarily to meet one condition as economically as possible and then be adapted to meet the second operating condition. At the same time, the installation should be designed with the possibility and in fact probability of future expansion of the facility.

Figure 3 indicates the cycle developed for the Manhattan Shelter project, the criteria of which called for the protection of 4 million day-time inhabitants of Manhattan Island for a buttoned-up period of up to 90 days duration. This was solely a civilian shelter type operation, planned for an 800-ft depth in the bedrock. As can be seen, this is an excellent example of the definition of utility interaction, the water being used in a series operation in sequence according to temperature requirements. If, for example, no advantage had been taken of the ability of 60° water to absorb sensible heat and the entire air-cooling requirements had been accomplished by mechanical refrigeration, the power requirements would have been increased by approximately 100,000 KVA and the water requirements by 156 million gallons per day. Water for showers, sewage conveyance, and cooking would increase these requirements. The total estimated cost of the utilities was thus reduced by between 70 and 80 million dollars.

It is realized, of course, that engineers who have been intimately connected with underground work will have recognized the effect of varying one utility at the possible expense of another for, after all, the prime consideration of any engineer is to accomplish the required end result for a minimum of client expenditure. The consideration of utility interaction is in the final analysis important in one main respect--dollars of construction cost compatible with required basic criteria.

POWER SOURCES

Let's look briefly at the major types of power sources available to us and the characteristics which affect their design for underground installations and coordination with other utilities. Figure 4 lists four power sources in order of increasing cost.

From the brief list of characteristics shown we can make the following observations:

Nuclear reactors are advantageous from the standpoint of protection because they do not require an air intake and exhaust and they can be completely isolated from the surface. They are a problem for two reasons:

1. at present they cost considerably more than other sources; and

2. except for the combustion steam plants, the heat which must be stored is much greater than the other systems listed.

There are two main reasons why nuclear reactor plants, as well as combustion steam plants, have so much more heat which must be stored or dissipated than diesel generators:

1. the efficiency of the diesel generators for small size plants is higher, although this is the smallest of the two effects; and

2. a diesel generator rejects only about half the waste heat to the cooling system. The remainder, ignoring small miscellaneous losses, goes out the exhaust.

In the reactor plant, practically all of the waste heat is rejected to the cooling system in the condenser and then must be dissipated or stored. The combustion steam plant rejects part of the waste heat up the stack.

We note in this figure also that the temperature at which the waste heat must be stored varies considerably. A diesel generator which uses water for cooling can reject heat to the water at any value up to the boiling point of water. Boiling must be avoided unless the diesel is designed for this type of cooling. If an intermediate heat exchanger is used for the diesels, the temperature of the reservoir cooling water may have to be limited to approximately 170° F.

In any steam cycle the efficiency of the cycle varies considerably with the temperature of cooling water available, as this determines the vacuum which can be maintained in the condenser. As the temperature of the cooling water goes up, the efficiency goes down. In practice one would not operate the steam turbine cycle with heat rejection approaching anywhere near to the 190° figure shown. It would be closer to 100°.

Combustion steam plants have the disadvantage of requiring large fuel storage and almost as much of a heat sink as the nuclear plant, and they also require air intakes and exhausts. Combustion steam plants have no advantages for underground installations except that they are conventional and present no operating problems. The gas turbines in some cycles require no cooling water, and therefore no heat sink but they require a lot of combustion air, approximately 4 to 5 times that required for a diesel.

Diesel generators require less of a heat sink than reactors, but they require fuel storage and air intakes and exhausts. They require the smallest capital investment and with their minimum heat sink requirements are the most advantageous power source at this time.

REFRIGERATION

In considering the means of refrigeration available we can compare the two types of refrigeration equipment normally used--mechanical refrigeration using the vapor cycle; and absorption refrigeration. Figure 5 shows the major items of interest for these cycles.

The absorption refrigeration cycle operating on the principle of using heat for its energy source has the advantage of requiring less installed electric generating capacity. Generally this cycle will be of advantage where waste heat is available, but as will be shown later, the high cost of heat sink capacity required for the greater heat rejection rate of the absorption cycle may offset any saving obtained from these lower power requirements.

In general it is very difficult, if not impossible, to predict which type of refrigeration will be most advantageous until a cycle analysis is performed for a particular installation.

Of course, there is never only one solution or one way to design utilities for an above-ground installation and this is equally true in an underground installation. The following examples show several solutions to a problem and the thinking behind the solution. It is not intended to imply that other solutions are not possible; nor is it intended to imply that these are the only problems to be encountered in utilities evaluation.

SPECIFIC EXAMPLES

Let us take a case where we will evaluate only two interacting utilities--power generation and air conditioning. We will assume we have to provide generating facilities for 1600 KW of load exclusive of refrigeration equipment, and that the refrigeration load is 600 tons. Let us assume that we are going to use diesel generators as the power generating equipment and see what kind of systems we can come up with.

As one solution, we will try using all-mechanical refrigeration. Figure 6 shows the heat balance that would result. Six hundred tons of refrigeration would require approximately 540 KW of electricity. Our generation is increased to 2140 KW. The waste heat from the diesels is about 6.41×10^6 Btu/hr, whereas from the refrigeration equipment it is 9×10^6 Btu/hr.

For normal operation we can use cooling towers to dispose of these waste heats. For "button-up" operation, we must assume that the cooling towers are out of operation immediately, and we must provide some means of disposing of this heat. We could have put these cooling towers underground, but this would require large amounts of air to be brought underground, which is a costly problem.

For "button-up" operation we would provide an underground reservoir, either water or ice. You will hear a paper on these reservoirs later today.

Now, let us look at another solution (Fig. 7). In an effort to reduce the electric load, we can try using absorption refrigeration. We have available in the exhaust gases about 3000 Btu/hr per kilowatt of electricity generated. This heat is at about 600°F. By use of heat recovery devices we can generate low-pressure steam. We will assume we will recover about 1000 Btu/KW-hr from the exhaust because we do not want to lower exhaust gases to too low a temperature, in order to prevent condensation in the exhaust. With the amount of heat available, we can support about 108 tons of absorption refrigeration. For the remaining 492 tons, we will have to use mechanical refrigeration.

Later we will tabulate data on all these cycles. But right now, we know that we have reduced the generating capacity required. We have also reduced the operating cost because less fuel is required. As an added cost we have heat recovery devices. We have less heat to dissipate from the diesels, more from the refrigeration equipment. We will have to wait until later to see what effect this has on the reservoir cost.

Figure 8 shows the case where we take the heat from the cooling system to use for absorption refrigeration. The reason is the same as in the previous case. We want to reduce the amount of electricity which must be generated, thus decreasing initial and operating cost. In this case we again support 108 tons of absorption refrigeration with the 3000 Btu/KW-hr given off to the cooling system, since we again assume about 1000 Btu of this amount can be recovered. Actually, commercial diesel cycles designed for steam cooling will produce about 2 lb of 15 psi steam/KW-hr when the exhaust heat exchanger is connected in series with the jacket circuit.

We shall see later what happens to the reservoir sizes, but we can note that in the case shown in Fig. 8 we are using heat which was going to have to be stored anyway; whereas in the case of using the exhaust gases, we recovered heat which was going out of the installation via the exhaust.

Figure 9 shows the cycle where we recover both the exhaust heat and the cooling heat for absorption refrigeration. We have reduced the amount of mechanical refrigeration to 395 tons but if we look at the heat rejected by refrigeration, we shall see that in this case it is a maximum.

Figure 10 tabulates the electric load of the four cases we have just looked at and also the amounts of mechanical and absorption refrigeration we have in each case.

In addition, Case V of Fig. 10 is shown as a hypothetical case where the entire available 6000 Btu/KW-hr is removed from the exhaust and jacket cooling system.

In the examples we have listed, as we increased the amount of absorption refrigeration we have decreased the amount of generating capacity we need. In the fourth case we need about 91 per cent of the first case, which was all mechanical refrigeration. This is a desirable situation as the first cost for the diesel and also the operating costs decrease due to less fuel required.

Case V is the ideal unattainable situation where the electric load has been reduced to 78 per cent of Case I.

In Fig. 11 we have tabulated the heat which in each case is rejected by the diesels in the cooling system and which is not used in refrigeration cycle and which therefore must be disposed of in some other way. We note that the amount decreases in each case.

The second column shows the heat which is rejected by the refrigeration machine and which must be disposed of. The amount increases in each case as we increase the amount of absorption refrigeration.

The third column shows the total heat which is rejected by each cycle. In the fourth column these values are tabulated in per cent of the first case values. We note that in each case it is more than the first case except for the cycle which uses only heat from the diesel cooling for absorption refrigeration. This is not unexpected, since we are using heat which would have to be stored anyway and we are getting some use from it.

In Cases II, IV, and V, even though we have reduced the amount of electricity which must be generated, we end up with more heat to store.

In the third case, before we jump to the conclusion that we have also decreased the problem of getting rid of the heat, we shall look at the last column. Here we have listed for each cycle the percentage of heat which must be stored at low temperature. We note that the per cent is least for the case of all-mechanical refrigeration.

As we have stated before, during normal operation this heat is rejected outside the facility, such as in cooling towers. During the button-up period, the heat must be stored within the facility. The size of this heat sink is affected by the temperature at which the heat is rejected.

Figure 12 shows the size and cost of the reservoirs required for these cycles for a 5-day and 30-day period, the cost of these reservoirs assuming a cost of $13.00 a cubic yard for excavation. For this figure we have assumed that the reservoir will be filled with water, with the initial temperature being 60° F. We have assumed no heat transfer to the rock. Later we will say something about using lower temperature for the reservoir and show the effect of heat transfer to the surrounding rock.

We note that in Case I the size of reservoir required for refrigeration equipment is four times as much as for the diesels, even though the amount of heat for the refrigeration is only 1.4 times as much as from the diesels. The reason is that we have assumed a maximum temperature of 170°F for the diesel reservoir and 100° F for the refrigeration reservoir. This means that with an initial temperature of 60° F the water in the diesel and refrigeration reservoirs can absorb 110 Btu and 40 Btu per pound, respectively. We use two reservoirs instead of one because the final temperatures permissible for the two purposes are quite different. If only one reservoir were used, the maximum temperature which could be used would be determined by the refrigeration equipment, and the total excavation would be much larger.

The size of reservoir required for the diesels is less in all the cases where we use some absorption refrigeration, and the size of the reservoir for refrigeration is more. The total excavation is more for each case where absorption refrigeration is used, even in the case where we used diesel cooling for absorption refrigeration, the case where we say that there is less total heat to be stored.

If we examine Cases I and IV, the first with all mechanical refrigeration, the latter with the maximum absorption refrigeration, we see that for a 5-day period the cost of reservoir is $56,000 more for the last case. If the criterion is to operate buttoned-up for 30 days the difference in the cost is $340,000. If we remember, Case IV required 91 per cent of the electric generation that Case I did. The economic choice of system for this example is influenced not only by the two operating conditions, normal and "buttoned-up" but also upon the length of time for which the installation must be designed for "buttoned-up" operation. The costs shown must be balanced against reduced costs of the diesel installation, including air shafts and fuel storage, and heat recovery devices.

Figure 13 shows the effect of using a lower initial temperature for the reservoirs. Unless we do something to precool the reservoirs, the reservoirs will assume the ambient temperature of the rock, which of course varies according to location. In our example we used 60° F.

We can precool the reservoirs and maintain them at 40° F by using our standard refrigeration machine for air conditioning the installation. We can see that the effect is to decrease the amount of excavation required for the reservoirs. It has a greater percentage effect upon the size of the refrigeration reservoir because of the fact that a lower maximum temperature is permissible for this reservoir.

By using an ice reservoir, we can decrease the size still more, and for two reasons. One is that we have a high heat capacity for unit reservoir volume. The other reason is that until the refrigeration reservoir temperature reaches 45° F or so, we can use the water directly as chilled water and we need not run the refrigeration machinery. During this period we need generate less electricity so that we save on the diesel reservoir also. In the example we have used a 40 per cent ice-to-water ratio by weight. We see that there is an appreciable saving which can be obtained. Of course we must furnish ice-making machinery, but the capacity required is small.

Figure 14 shows the effect of heat transfer to the rock. The ordinates are plotted as the ratio of heat capacity of the reservoir with heat transfer to rock, to heat capacity of the reservoir with no heat transfer to rock. The abscissa is the length of time in days that heat is rejected to the reservoir. A constant rate of heat input is assumed. Curves are shown for several sizes of tunnels which are assumed to be of rectangular cross-section. The curves are plotted using heat transfer equations developed by the Bureau of Standards.

We note that with a small cross-section we get appreciably more effect from the heat capacity of the rock than we do for the larger cross-section, as can be expected because we have more surface for heat transfer. Also, the time for which the reservoir must operate has a great effect upon the benefit we can get.

Although we would like to have small tunnels, the actual size selected is again based on economics, because the small cross-sections cost much more to excavate than the larger ones, per unit volume.

To see what benefit we may expect from heat transfer to the rock, we see that with a given temperature rise, for a 20 x 20 ft tunnel we expect to have about 15 per cent more heat capacity in the reservoir for a 5-day period and 45 per cent for a 30-day period.

In conclusion, we should point out that cycles and comparisons used in this paper were kept relatively simple as an aid to ease of understanding. In an actual evaluation the effect of all utilities on the entire system should be analyzed and the whole problem approached as an entity considering the effect of each on the other.

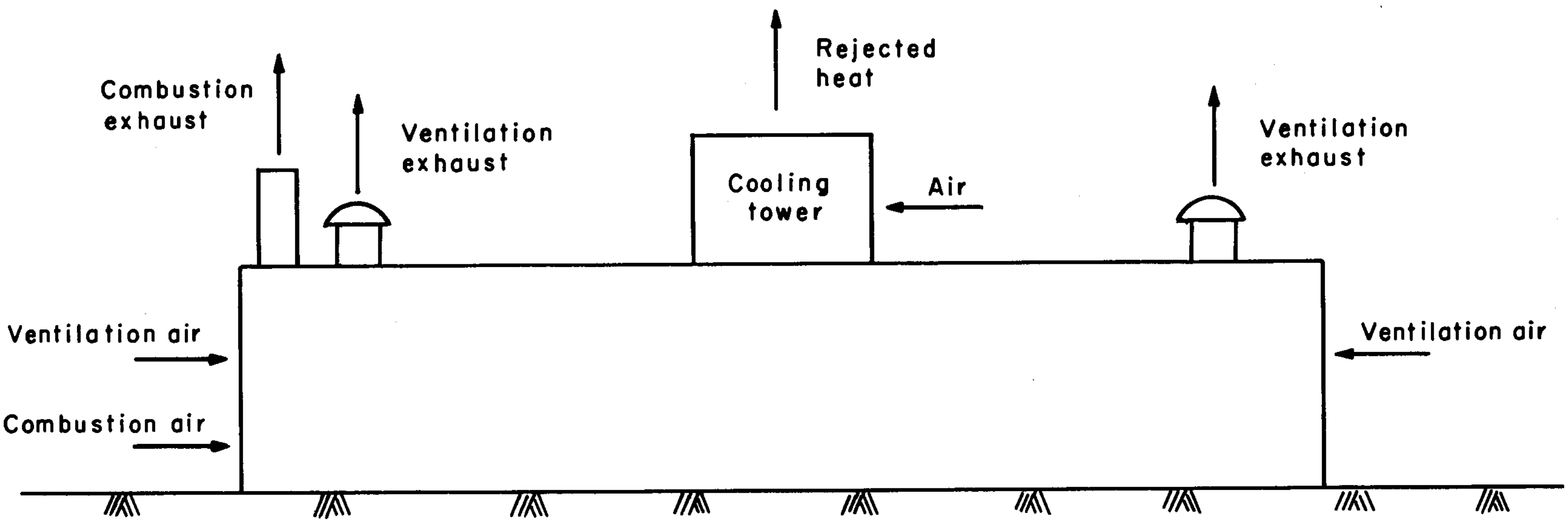


Fig. 1—Conventional above ground installation

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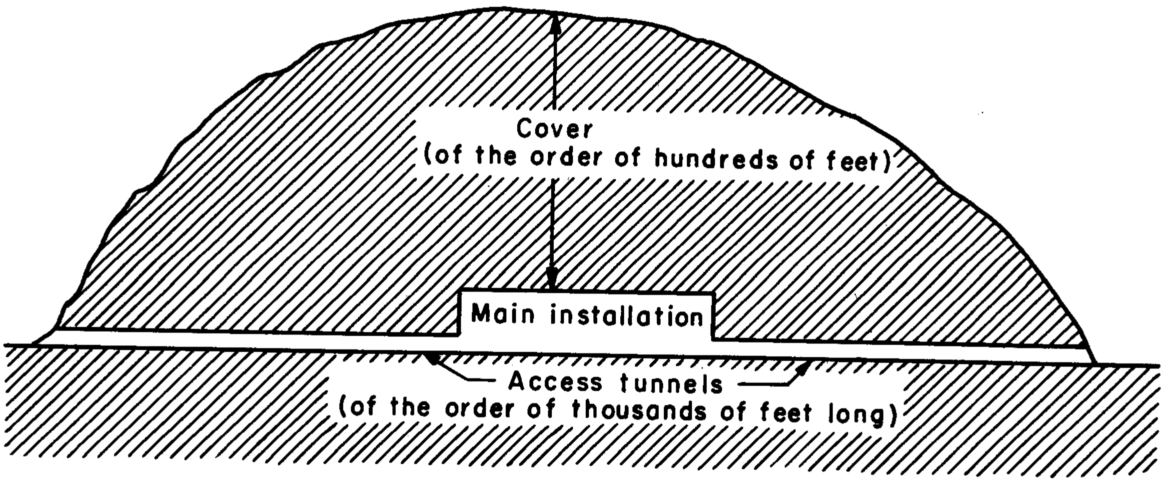

Fig. 2 — A type of installation

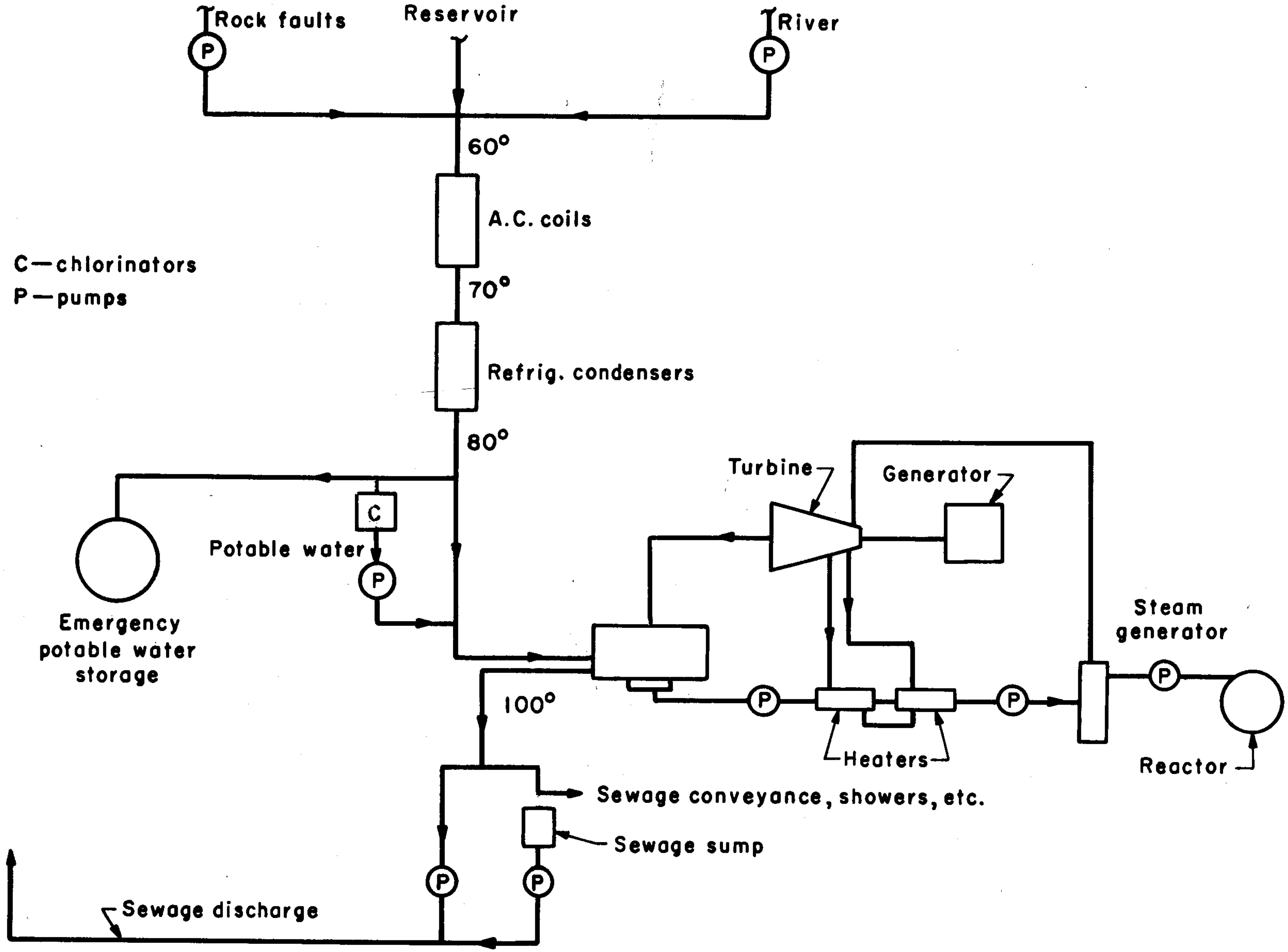

Fig. 3—Diagrammatic arrangement of utilities

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Fig. 4

CHARACTERISTICS OF POWER GENERATORS

	Cycle Efficiency %	Air Requirements CFM/KW	Heat Rejected to Cooling System Btu/KW-Hr	Temp. at Which Heat is Rejected to Cooling System
Diesel Generators	~ 30	3-5	~ 3,000	~ 170° F.
Gas Turbines	19-30	10-20	0	-
Combustion-Steam Plants	17-25	2.5-3.8	8,820-15,300	80°-190° F.
Nuclear Reactor-Steam Plant	17-23	0	10,650-15,900	80°-190° F.

Fig. 5

CHARACTERISTICS OF REFRIGERATION

	Type of Energy Required	Amount	Heat Rejected to Cooling Water	Temp. at Which Heat is Rejected
Absorption Refrigeration	Steam or Hot water	~19,000 Btu/ton	~31,000 Btu/ton	~100° F.
Mechanical Refrigeration	Mechanical work	~ 0.9 KW/ton	~15,000 Btu/ton	~100° F.

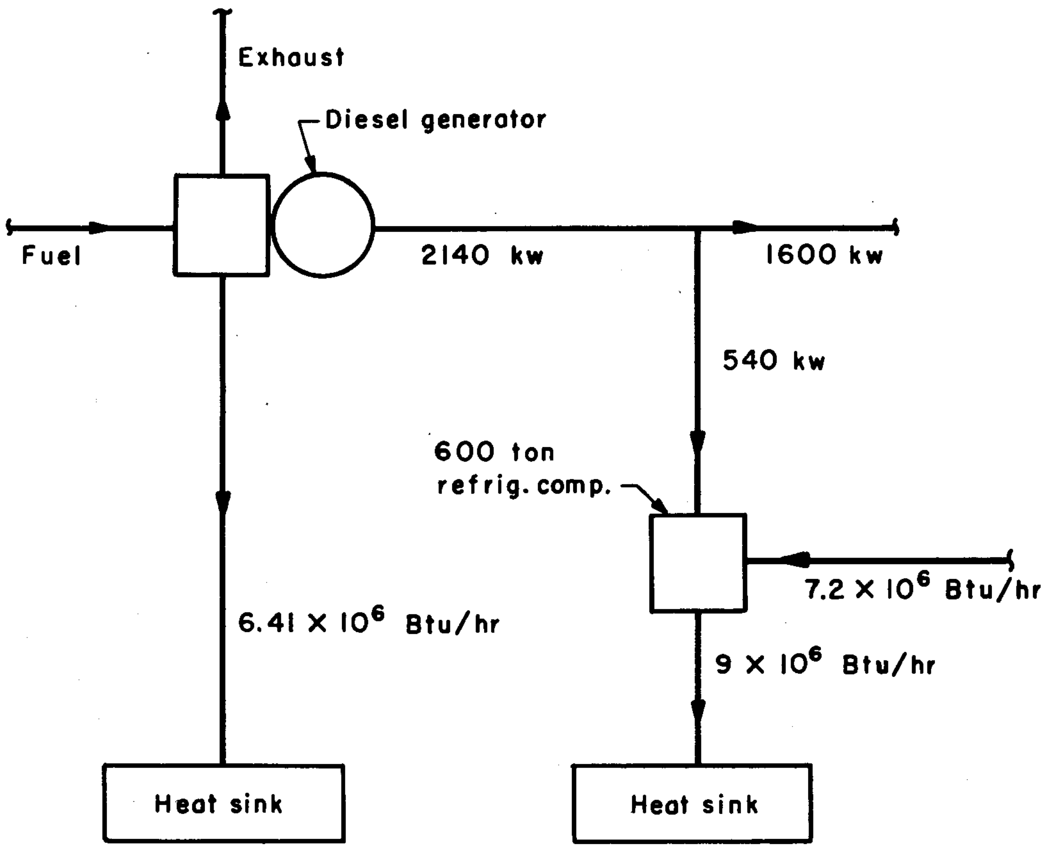

Fig. 6—Cycle with all mechanical refrigeration

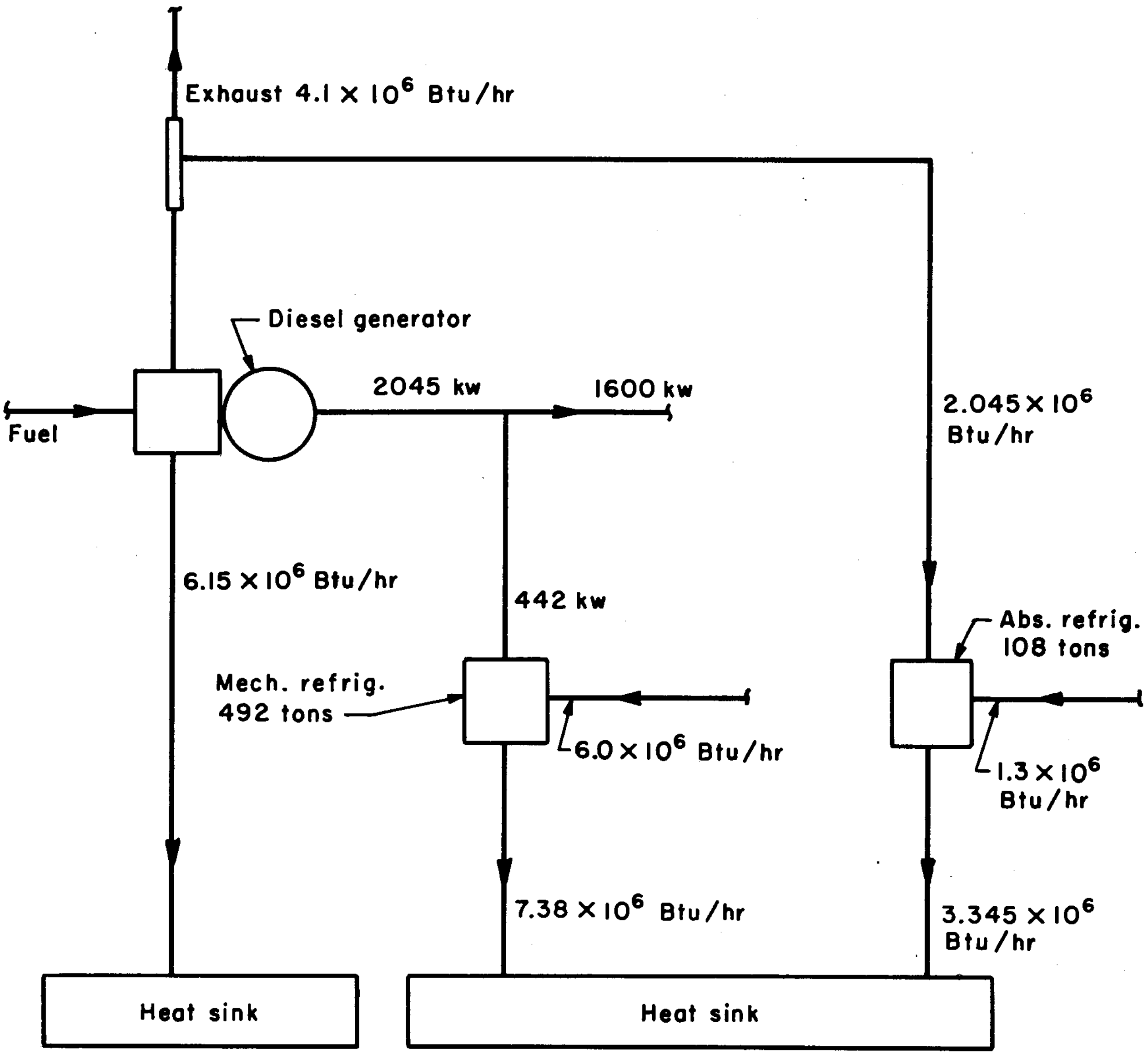

Fig. 7 — Cycle using exhaust heat for absorption refrigeration

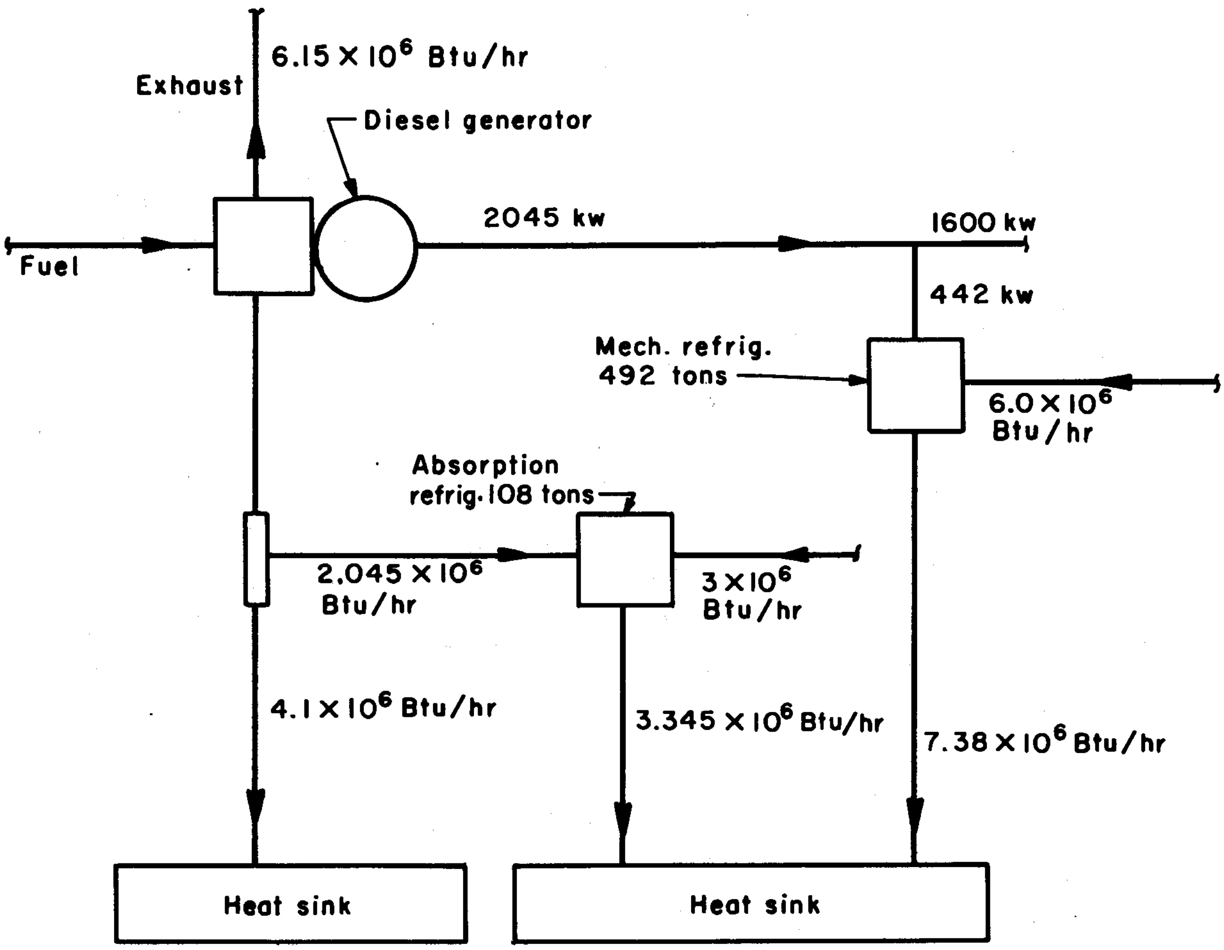

Fig. 8 — Cycle using diesel cooling for absorption refrigeration

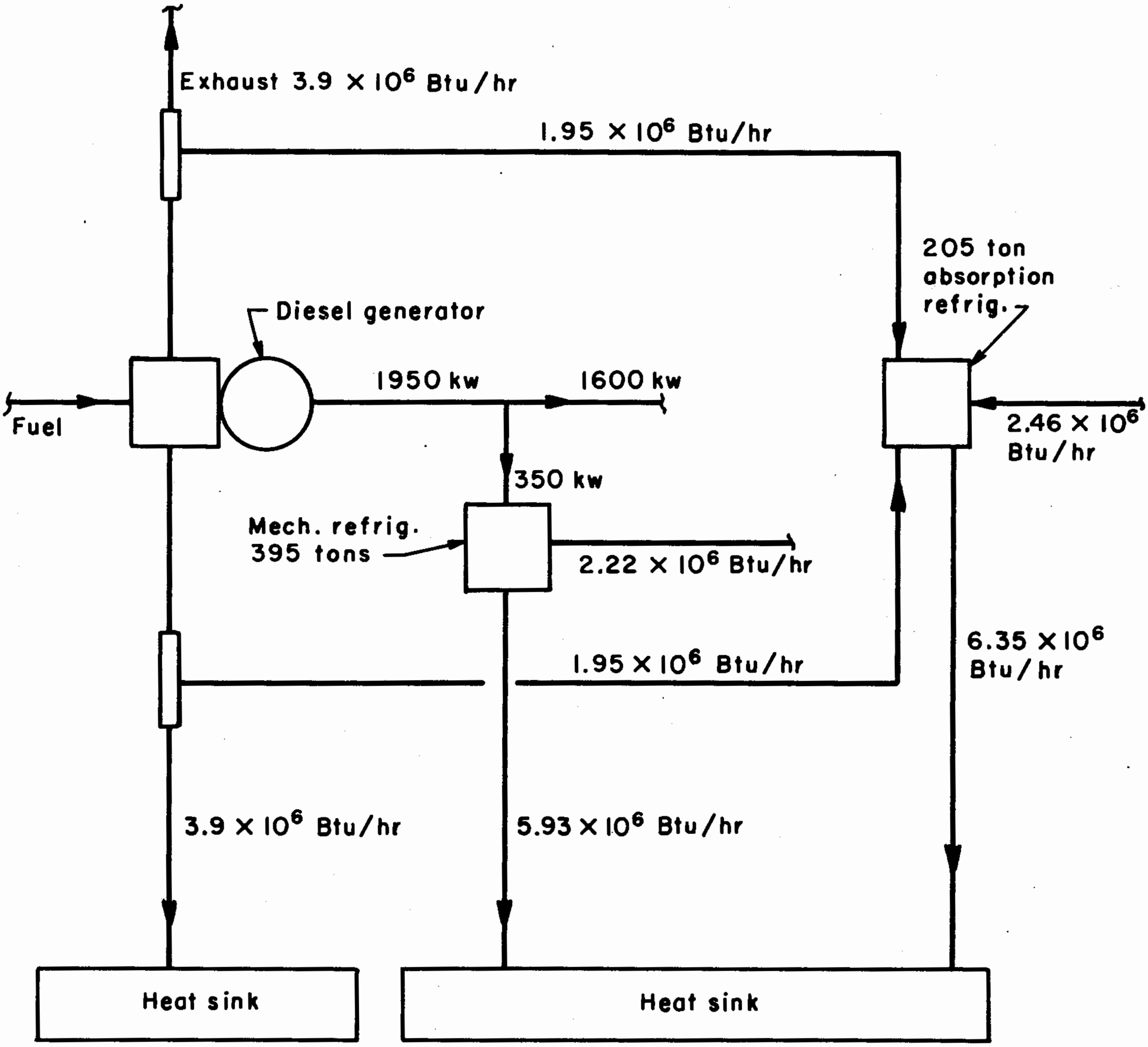

Fig. 9 — Cycle using diesel exhaust and cooling for absorption refrigeration

Fig. 10

ELECTRICAL AND REFRIGERATION LOADS FOR VARIOUS CYCLES

		Diesel Generator Load KW	Electric Load % of Case I	Mechanical Refrigeration Tons	Absorption Refrigeration Tons
CASE I	All Mechanical Refrigeration	2140	100%	600	0
CASE II	Using Exhaust for Absorption Refrigeration	2045	95.9	492	108
CASE III	Using Diesel Cooling for Absorption Refrigeration	2045	95.9	492	108
CASE IV	Using Exhaust and Cooling for Absorption Refrigeration	1950	91.0	395	205
CASE V	100% of all Diesel Exhaust and Cooling Waste Heat Used for Refrigeration	1668	78.0	74	526

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Fig. 11

HEAT REJECTED TO COOLING WATER IN VARIOUS CYCLES

Case	Rejected by Diesels Btu/hr.	Rejected by Refrigeration Btu/hr.	Total Heat Rejected Btu/hr.	Total Heat Rejected % of Case I	% of Heat Rejected at Low Temp.
I - All Mechanical Refrigeration	6.4×10^6	9×10^6	15.4×10^6	100%	58.5%
II - Exhaust Used for Absorption Refrigeration	6.15×10^6	10.75×10^6	16.9×10^6	110%	63.6%
III - Cooling Used for Absorption Refrigeration	4.1×10^6	10.75×10^6	14.85×10^6	96.2%	72.4%
IV - Exhaust and Cooling Used for Absorption Refrigeration	3.9×10^6	12.28×10^6	16.18×10^6	105%	76.%
V - 100% of all Diesel Exhaust and Cooling Waste Heat Used for Refrigeration	0	17.46×10^6	17.46×10^6	113%	100.%

Fig. 12

RESERVOIR SIZE AND COST
(Water Initial Temperature $60^{o}F$)

		Size for 5 Days Cu.Yds.	Size for 30 Days Cu.Yds.	Cost for 5 Days @ $13./CY	Cost for 30 Days @ $13./CY
CASE I	All Mechanical Refrigeration				
	Diesel Reservoirs	4,000	24,000		
	Refrigeration Reservoirs	16,000	96,000		
	Total	20,000	120,000	$260,000.	$1,560,000.
CASE II	Exhaust Used for Absorption Refrig.				
	Diesel Reservoirs	3,950	23,500		
	Refrigeration Reservoirs	19,000	114,200		
	Total	22,950	137,700	$298,000.	$1,785,000.
CASE III	Diesel Cooling Used for Absorption Refrigeration				
	Diesel Reservoirs	2,600	15,600		
	Refrigeration Reservoirs	19,000	114,000		
	Total	21,600	129,600	$281,000	$1,685,000.
CASE IV	Exhaust and Cooling Used for Absorption Refrigeration				
	Diesel Reservoirs	2,520	15,100		
	Refrigeration Reservoirs	21,800	131,000		
	Total	24,320	146,100	$316,000	$1,900,000.
CASE V	100% Waste Heat Used				
	Diesel Reservoirs	-	-		
	Refrigeration Reservoirs	31,000	186,000		
	Total	31,000	186,000	$402,000.	$2,420,000.

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Fig. 13

EFFECT OF RESERVOIR INITIAL TEMPERATURE

CASE I - DIESELS, ALL MECHANICAL REFRIGERATION

(5-Day Period)

Reservoir Initial Conditions	Diesel Reservoir Cu.Yds.	Refrig. Reservoir Cu.Yds.	Total Reservoirs Cu.Yds.	Cost of Reservoirs at $13/CY
60° F. Initial Temperature	4,000	16,000	20,000	$260,000.
40° F. Initial Temperature	3,530	10,650	14,180	184,000.
Ice Reservoir (40% Ice to Water Ratio by Wt.)	2,050	4,630	6,680	87,000.

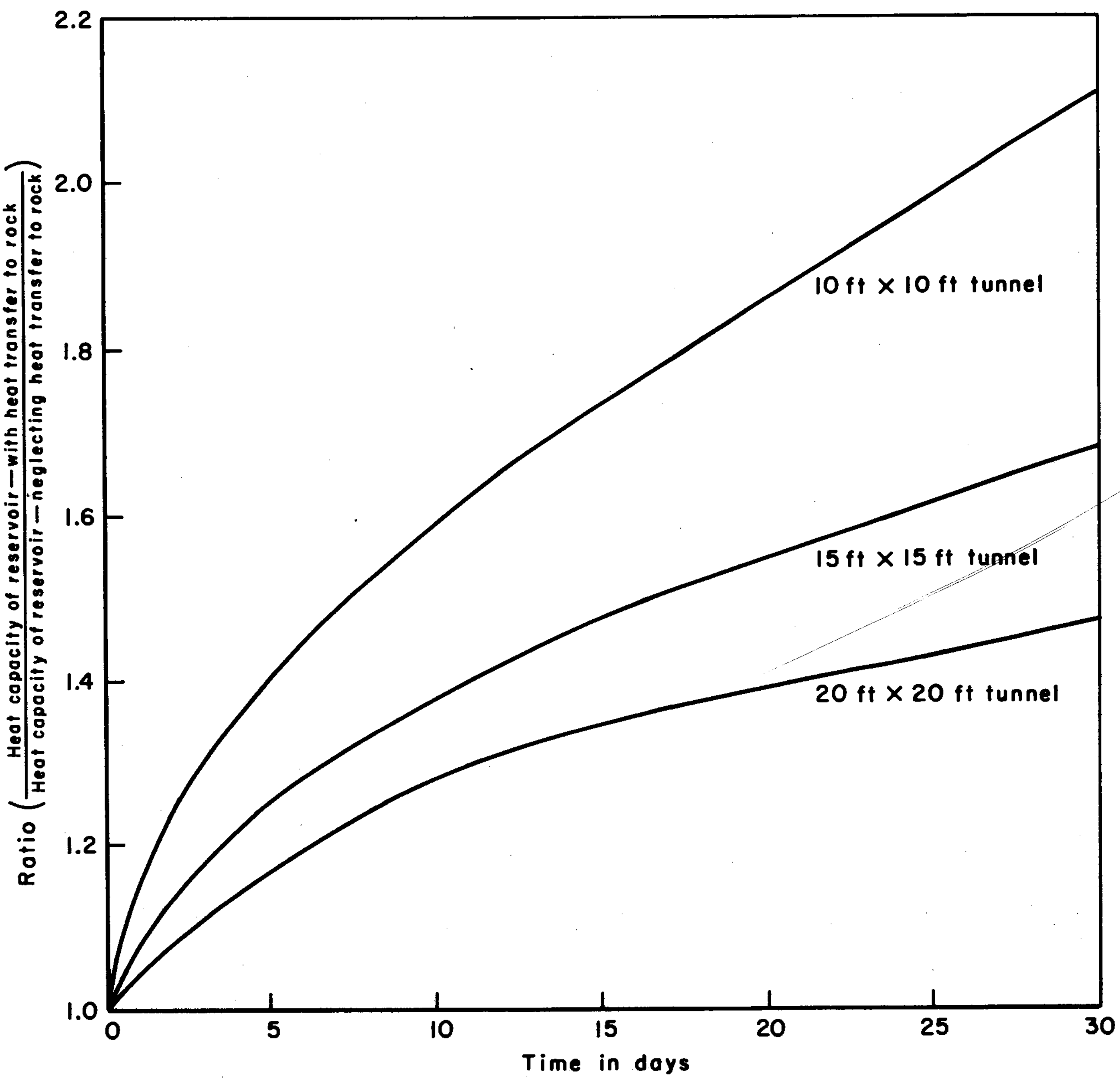

Fig. 14 — Effect of rock on heat capacity of reservoirs